Geotechnical Engineering Report

GREEN VALLEY 3 APARTMENTS

WKA No. 13081.02

May 4, 2021

(revised February 16, 2022)

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GREEN VALLEY 3 APARTMENTS
4840 Business Center Drive
Fairfield, California
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INTRODUCTION

We have completed a geotechnical engineering study for the proposed apartment complex to be constructed at 4840 Business Center Drive in Fairfield, California (see Figure 1). The purposes of our study have been to explore the existing soil and groundwater conditions across the planned apartment complex site, and to provide geotechnical engineering conclusions and recommendations for use in design and construction of the proposed apartment complex. This report presents the results of our study. The report has been revised to reflect the current apartment complex and parking structure layout.

Scope of Services

Our scope of services for this project has included the following tasks:

1. Site reconnaissance;
2. Review of a previous geotechnical engineering study performed for a property in the vicinity of the site;
3. Review of a United States Geological Survey (USGS) topographic map, historical aerial photographs, and available groundwater information relevant to the site;
4. Subsurface explorations, including the drilling and sampling of seven borings to depths ranging from approximately 16½ to 51½ feet below existing site grades. We also advanced four cone penetrometer test (CPT) soundings to depths ranging from about 41 to 101 feet below existing site grades;
5. Collection of representative bulk samples of surface and near-surface soils;
6. Laboratory testing of selected soil samples;
7. Engineering analyses; and,
8. Preparation of this report.

Supplemental Information

Our office also prepared a separate *Phase I Environmental Site Assessment* report for the site (WKA No. 13081.01, dated March 16, 2021). Information from this environmental report was used during the preparation of this report.

Supplemental information reviewed for this study also included the following report prepared by our firm for the AG Spanos Companies: Wallace-Kuhl & Associates, *Geotechnical Engineering Report* (WKA No. 11731.01, dated March 12, 2018), prepared for the Green Valley II Apartments project located about ½-mile northeast of the site.

Figures and Attachments

This report contains a Vicinity Map as Figure 1; a revised Site Plan showing the planned apartment complex layout and approximate locations of the explorations performed at the site as Figure 2; and, Logs of Soil Borings as Figures 3 through 9. An explanation of the symbols and classification system used on the logs is included as Figure 10. A map showing the nearest designated Alquist-Priolo Earthquake Fault Study Zones (*Special Study Zones*) to the site is included as Figure 11.

Appendix A contains general information regarding project concepts, exploratory methods used during our study, and laboratory test results not included on the boring logs. Appendix B contains copies of the CPT reports provided by Middle Earth Geo Testing, Inc. of Hayward, California. Appendix C contains copies of the output files for our liquefaction analysis. Appendix D contains plots of lateral loading of ACIP/APGD piles for “free” and “fixed” head conditions. Appendix E contains *Guide Earthwork Specifications* that may be used in preparation of contract documents. Appendix F contains *Guide Specifications for Auger Cast-in-Place (ACIP) Piles* that also may be used in the preparation of contract documents.

Proposed Development

Based on review of a drawing titled “*Floor Plan Level 02*,” dated August 12, 2021, prepared by Kephart (project architect), and our discussions with Mr. Nicolas Ruhl of the Spanos Corporation, the project will include construction of a multi-story parking garage, a four 4-story apartment building, and a two-story clubhouse/leasing building. The planned locations of the apartment building, clubhouse, and parking structure are presented on Figure 2. We anticipate the buildings will include an elevator system; however, below-grade basements are not planned for this project. We understand the apartment and clubhouse/leasing buildings will be constructed of wood-framing with interior slab-on-grade lower floors (potentially post-tensioned [PT] slab foundations). The parking structure will be constructed of reinforced concrete.

Based on the planned construction described above, structural loads are anticipated to be moderately heavy for the apartment and clubhouse/leasing buildings and relatively heavy for the parking structure. Associated development will include construction of underground utilities, landscaped areas, exterior flatwork, a below-grade swimming pool, parking areas and drive aisles.

We understand the buildings will be structurally tied to each other across expansion joints. For the purposes of this study, we have assumed that the expansion joints will not act as seismic isolation joints and will not serve as a damping system; however, this should be confirmed by the project structural engineer. Please note that based on review of the 2019 *California Building Code (CBC)* and Section 11.4.8 of *American Society of Civil Engineers (ASCE) 7-16*, seismically isolated structures and structures with damping systems require a site-specific ground motion hazard analysis.

A grading plan was not available for review at the time this report was prepared; however, based on the existing site topography, our understanding of the planned development and the existing subsurface soil conditions, we anticipate cuts and fills on the order of one to five feet will be required for development of the site, with the exception of excavations required for construction of underground utilities and the swimming pool. Excavations up to six to 10 feet in depth are anticipated for construction of the underground utilities and the swimming pool.

FINDINGS

Site Description

The site is located at 4840 Business Center Drive in Fairfield, California. The site is located on about 5.8 acres of land identified as Solano County Assessor's Parcel Number 0148-540-350. The site is bounded to the northeast by commercial development; to the southeast by vacant land covered with vegetation, beyond which is Business Center Drive; to the southwest by dense tree/bush cover, beyond which is Green Valley Road and Green Valley Creek; and, to the northwest by dense tree/bush cover and residential development.

Our field explorations were performed on March 16 and 17, 2021. During this time the site was generally covered with a dense growth of green grass. Two rough-graded pads were observed at the site. One pad was observed in the north portion of the site in southwest-northeast orientation. The other pad was observed in the south portion of the site in northwest-southeast orientation. We are not aware of, and have not been provided with, a previous geotechnical engineering report(s) for the site or any compaction testing records regarding the existing building pad fills.

The eastern boundary of the site was paved with asphalt concrete and used as a driveway. The western boundary of the site was covered with a dense growth of bushes and trees. Overhead utilities were observed in the southwest portion of the site. Evidence of underground utilities was observed along west and east boundaries of the site.

The rough-graded pads and surrounding areas are relatively level. Based on topographic information shown on the *ALTA/NSPS Land Title Survey*, prepared by TSD Engineering, Inc., dated March 15, 2021, the surface elevations of the rough-grade pads range from about +19 to +20 feet National Geodetic Vertical Datum of 1929 (NGVD 29). The surface elevations of the areas surrounding the pads range from about +15 to +18 feet NGVD 29. These elevations are consistent with the *United States Geologic Survey (USGS) Topographic Map of the Cordelia Quadrangle*, dated 2018.

Historical Aerial Photograph Review

We reviewed historical aerial photographs showing the planned apartment complex site available from Environment Data Resources, Inc., and Google Earth software. Available photographs were taken in the years 1937, 1947, 1952, 1968, 1974, 1982, 1993, and 2002 through 2020.

Review of the photograph taken in 1937 shows the site as vacant land covered with vegetation. Review of the photographs taken in 1947, 1952, 1968, 1974, 1982, and 1993 show the site has generally remained unchanged since at least 1937. Review of the photograph taken in 2002 shows grading activities in the major portion of the site. It appears the rough-graded pads observed at the site were constructed during this time. The north boundary of the site remains covered with vegetation. Review of the photograph taken in 2003 shows what appears to be the rough-graded pads observed at the site. The pads and surrounding areas are shown to be covered with vegetation. Review of the remaining photographs show the site has generally remained unchanged since at least 2003. The paved driveway observed along the east boundary of the site appears to have been constructed in 2006.

Subsurface Soil Conditions

On March 16 and 17, 2021, seven borings (D1 through D7) and four CPT soundings (CPT1, CPT1A, CPT2 and CPT3) were performed at the site at the approximate locations shown on Figure 2. The boring locations were selected based on the conceptual site plan provided at that time but still provide adequate coverage of the building areas based on the current development plan.

Based on the borings, the surface and near-surface soil conditions at the site generally consist of soft to very stiff, high plasticity, fat clay with variable amounts of sand to depths ranging from about six to 25 feet below existing site grades. Based on the borings and CPT, these soils are underlain by alternating layers of relatively stiff to hard, moderate plasticity, lean clay with variable amounts of silt and sand and relatively loose to dense, silty and clayey sands with variable amounts of gravel to the explored depths ranging from about 16½ to 101 feet below existing site grades. Please note that CPTs 1, 1A and 3 were terminated at depths ranging from about 41 to 45 feet below existing site grades due to practical cone refusal on a relatively hard/very dense soil layer. The relatively hard/dense soil layer was observed in CPT2 and Boring D3 at similar depths; however, practical cone/auger refusal was not encountered in these explorations. The soil conditions encountered in the explorations performed at the site are consistent with those encountered during the field exploration included in the 2018 *Geotechnical Engineering Report* (WKA No. 11731.01) referenced in this report, prepared for a property located about ½-mile northeast of the site (Green Valley II Apartment project).

At the completion of the field exploration activities, the explorations were backfilled with a slurry of neat cement, bentonite, and water in accordance with Solano County Department of Resource Management Environmental Health Services requirements. For specific information regarding the soil conditions at a specific exploration location, please refer to the Logs of Soil Borings, Figures 3 through 9, and the CPT sounding reports included in Appendix B.

Groundwater

Groundwater was observed in Borings D1 through D7 on March 16 and 17, 2021 at depths ranging from approximately seven to eight feet below existing site grades. Please note the borings may not have been left open long enough for groundwater to reach static equilibrium.

To supplement our groundwater data, we reviewed available groundwater information at the California Department of Water Resources (DWR) website. The DWR periodically monitors groundwater levels in wells across the state. Their website shows a well located approximately three-quarters (¾) of a mile southeast of the site. The well is identified as Well No. 04N02W07D001M with a ground surface elevation of about +17 feet NGVD 29. Groundwater data for this well was recorded from June 7, 1949 to at least October 4, 1972. Data shows the highest recorded groundwater elevation was about +15 feet NGVD 29 at the well (about two feet below the ground surface at the well location) on March 10, 1969. The lowest recorded groundwater elevation was about -5 feet NGVD29 at the well (about 22 feet below the ground surface at the well location) on September 29, 1949.

The groundwater conditions observed in our borings are within the historical groundwater level range described above. Based on this data, for the purposes of this study, we have assumed the historical high groundwater at the site to be about two feet below lowest portion of the site or about +13 feet NGVD 29.

CONCLUSIONS

Seismicity and Faults

As shown in Figure 11, the site is not located within an Alquist Priolo Earthquake Fault Study Zone, which are established around faults known to be active within the past 11,700 years. However, the site located within close proximity of several surface faults that are presently zoned as active or potentially active by the California Geological Survey (CGS) pursuant to the guidelines of the Alquist-Priolo Earthquake Fault Zoning Act. The nearest Alquist-Priolo Earthquake Fault Study Zone has been established around faults associated with the Cordelia fault zone. The edge of this fault zone is located about 700 feet east of the site, also as shown in Figure 11.

Review of the *Fault Activity Map of California*, dated 2010 and prepared by the CGS, shows there are several active faults located within a 25-mile radius of the site. The most notable active faults in the vicinity of the site are those associated with the Cordelia fault zone, located less than a ¼-mile east of the site; the Green Valley fault zone, located less than one mile west of the site; the West Napa fault zone, located about five to 10 miles west of the site; the Concord fault zone, located about 10 to 15 miles south of the site; the Rodgers Creek fault zone, located about 20 miles west of the site; the Hayward fault zone, located about 20 miles southwest of the site; and, the Greenville fault zone, located about 20 miles south to southeast of the site. The epicenter for the August 24, 2014 earthquake on the West Napa Fault was located about 10 miles west of the site. Based on this information, the potential for the site to experience significant ground shaking from future earthquakes is relatively high.

Seismic Site Class

Shear wave velocities obtained at location CPT2 varied from about 488 to 1,354 feet per second (fps) within approximately the upper 100 feet of the soil profile. The average shear wave velocity within the upper 100 feet at CPT2 was determined to be about 852 fps, in accordance with Section 1613.3.2 of the 2019 edition of the *California Building Code* (CBC) and Chapter 20 of *American Society of Civil Engineers (ASCE) Standard 7-16*.

Based on Table 20.3-1 of ASCE 7-16, a seismic site Class D applies to sites with average shear wave velocities between 600 to 1,200 fps for the upper 100 feet of the ground surface. Therefore, according to the information obtained from the shear wave velocity measurements at

CPT2, the soils at this site can be designated as Site Class D in determining seismic design forces for this project.

2019 CBC/ASCE 7-16 Seismic Design Criteria

The 2019 edition of the *CBC* references *ASCE Standard 7-16* for seismic design. Based on field test results, review of the CPT data, and our experience in the local area, in our opinion the site can be designated as Site Class D in determining seismic design forces for this project. The seismic design parameters provided in Table 1 were determined based on a Site Classification D and the latitude and longitude for the central portion of the site using the web interface developed by the *Structural Engineers Association of California* and *California's Office of Statewide Health Planning and Development*. Since S_1 is greater than 0.2 g, the coefficient values F_v, S_{M1}, and S_{D1} presented in Table 1 are valid for this project, provided the requirements in Exception Note No. 2 of Section 11.4.8 of *ASCE 7-16* apply. If not, a site-specific ground motion hazard analysis is required for this project.

Table 1: 2019 CBC/ASCE 7-16 Seismic Design Parameters

Latitude: 38.2205° N Longitude: 122.1413° W	ASCE 7-16 Table/Figure	2019 CBC Table/Figure	Factor/Coefficient	Value
0.2-second Period MCE	Figure 22-1	Figure 1613.2.1(1)	S_S	1.500 g
1.0-second Period MCE	Figure 22-2	Figure 1613.2.1(2)	S_1	0.600 g
Soil Class	Table 20.3-1	Section 1613.2.2	Site Class	D
Site Coefficient	Table 11.4-1	Table 1613.2.3(1)	F_a	1.00
Site Coefficient	Table 11.4-2	Table 1613.2.3(2)	F_v	1.700*
Adjusted MCE Spectral Response Parameters	Equation 11.4-1	Equation 16-36	S_{MS}	1.500 g
	Equation 11.4-2	Equation 16-37	S_{M1}	1.020 g*
Design Spectral Acceleration Parameters	Equation 11.4-3	Equation 16-38	S_{DS}	1.000 g
	Equation 11.4-4	Equation 16-39	S_{D1}	0.680 g*
Seismic Design Category	Table 11.6-1	Table 1613.2.5(1)	Risk Category I - IV	D
	Table 11.6-2	Table 1613.2.5(2)	Risk Category I - IV	D

Notes: MCE = Maximum Considered Earthquake; g = gravity
* The value is valid provided the requirements in Exception Note No. 2 in Section 11.4.8 of ASCE 7-16 are met. If not, a site-specific ground motion hazard analysis is required.

Liquefaction Potential

Liquefaction is a soil strength and stiffness loss phenomenon that typically occurs in loose, saturated cohesionless soils as a result of strong ground shaking during earthquakes. The potential for liquefaction at a site is usually determined based on the results of a subsurface geotechnical investigation and the groundwater conditions beneath the site. Hazards to buildings associated with liquefaction include bearing capacity failure, lateral spreading, and differential settlement of soils below foundations, which can contribute to structural damage or collapse.

The findings of the explorations performed at the site revealed the underlying soils generally consist of relatively soft to very stiff, high plasticity, fat clay with variable amounts of sand to depths ranging from about six to 25 feet below existing site grades, underlain by alternating layers of relatively stiff to hard, moderate plasticity, lean clay with variable amounts of silt and sand, and relatively loose to dense, silty and clayey sands with variable amounts of gravel to the explored depths ranging from about 16½ to 101 feet below existing site grades. For the purposes of this study, we have assumed the historical high groundwater at the site to be about two feet below lowest portion of the site or about +13 feet NGVD 29.

Typically, the relatively stiff, cohesive, and relatively dense, granular soil conditions revealed by the explorations performed at the site are resistant to liquefaction during earthquake ground shaking. However, we have performed a site-specific liquefaction analysis as part of this study to determine factors of safety against liquefaction using the representative soil conditions encountered at the four CPTs performed at the site (CPT1, CPT1A, CPT2 and CPT3) and the assumed historical high groundwater elevation.

Liquefaction Analysis and Results

The potential for liquefaction at the site was evaluated using data from the four CPTs performed at the site (CPT1, CPT1A, CPT2 and CPT3) and the soil liquefaction assessment software CLiq (Version 2.2.1.4), which was developed by GeoLogismiki. The software utilizes data collected from CPT soundings to determine factors of safety against liquefaction for varying earthquake input energies and uses the results of the National Center for Earthquake Engineering Research (NCEER) liquefaction evaluation methods summarized by Youd, et al, 2001. Input values were obtained using the results of the four CPTs referenced above. Based on our review of historical groundwater data relevant to the site, a design (“historical high”) groundwater level of six feet below existing site grades (or about +13 feet NGVD 29) was used in our liquefaction analysis. The mapped geometric mean peak ground acceleration (PGAM) of 0.60 g, determined in accordance with Equation 11.8-1 of ASCE Standard 7-16, was used in our analysis. A mode magnitude earthquake of 6.7, determined using the 2014 USGS National Seismic Hazard Mapping Project (NSHMP) Probabilistic Seismic Hazard Analysis (PSHA) Interactive Deaggregation website, also was used in our analysis.

Both because the case histories of liquefaction and its effects on which simplified procedures are based on are heavily weighed to clean sands and because of concern that CPT soundings are not fully drained in sandy silt soils, the Ic (Soil Behavior Type Index) cutoff was conservatively set at 2.15 (an Ic value of 2.05 is the boundary between normalized soil behavior types [SBTn] 5 [silty sands to sandy silts] and 6 [sands to silty sands] [Dr. Robert Pyke, correspondence, 2008]).

The results of our liquefaction analysis indicate isolated granular soil layers of various thicknesses between the approximate depths of about 25 to 39 feet below existing site grades encountered at the four CPTs referenced above have a factor of safety against liquefaction below 1.3. A factor of safety below 1.3 requires a liquefaction-induced settlement analysis.

Liquefaction potential at the site was also evaluated based on the Liquefaction Potential Index (LPI). The LPI is a measure of the liquefaction potential based on an analysis of the entire vertical soil profile not just discrete layers (Iwasaki, 1986; Toprak and Holzer, 2003). Factors taken into consideration for the LPI calculations include: thickness of the liquefied layer; proximity of the liquefied layer to the surface; and, the factor of safety. The LPI ranges from 0 to 100 with the value zero representing no liquefaction potential. Surface manifestations of liquefaction occur at LPI ≥ 5. The LPI for the soil conditions at the four CPTs referenced above was calculated to be less than 2, **indicating the risk of liquefaction at the site is "very low" during the design seismic event** (mode magnitude earthquake of 6.7 and a PGA_M of 0.60 g).

Seismic Induced Settlement

Based on the results for the liquefaction analysis, we evaluated the four CPTs referenced above for post-liquefaction settlement using the methodology of Ishihara and Yoshimine (1992). The results of our liquefaction analysis indicate the calculated seismic settlements at the four CPTs referenced above range between about ¼ of an inch and ½ of an inch, and in our opinion can be considered negligible. Copies of the output files for the liquefaction analysis are provided in Appendix C.

General

To our knowledge there have been no reported instances of liquefaction having occurred within the local area during the major earthquake events of 1892 (Vacaville-Winters), 1906 (San Francisco), 1989 (Loma Prieta), and 2014 (West Napa/American Canyon). Based on the soil conditions encountered at the site and our liquefaction analysis, including LPI evaluations, **it is our professional opinion that the potential for liquefaction of the soils beneath the site is relatively low if the site experiences significant ground shaking during an earthquake.**

We do not anticipate the seismic settlements calculated will adversely affect the performance of the planned structures and associated improvements from a life-safety perspective, provided the recommendations included in this report update regarding site clearing, subgrade preparation and engineered fill placement are carefully followed.

Bearing Capacity and Building Support

Our work revealed that approximately the upper one to two feet of soil across the site are in a relatively soft condition, likely due to the surface and near-surface soils at the site being subjected to several years of seasonal drying and wetting. The surface and near-surface clay soils at the site also are considered to have the potential to cause vertical movements of lightly loaded conventional foundations (continuous and/or isolated spread footings), interior floor slabs, exterior flatwork, and pavements. In our opinion, these soils should not be relied upon to support structural improvements associated with the apartment complex in their current condition.

Over-excavation, processing and compaction of the surface and near-surface soils in accordance with the recommendations of this report will be required so that surface and near-surface soils are capable of adequately supporting lightly loaded conventional foundations, slab-on-grade concrete and pavements associated with the apartment complex. Specific recommendations to over-excavate, scarify, moisture condition, and compact the surface and near-surface soils have been provided in the Subgrade Preparation section of this report.

Based on our field investigation and laboratory test results, it is our opinion the undisturbed native soils and engineered fill, properly placed and compacted in accordance with the recommendations of this report, are capable of supporting lightly loaded conventional foundations, slab-on-grade concrete and pavements associated with the apartment complex, provided the recommendations included in this report regarding site clearing, subgrade preparation, and engineered fill placement are carefully followed. However, untreated on-site or imported clays are not considered suitable for direct support of slab-on-grade concrete, including PT slab foundations, sidewalks and pool decks.

Field and laboratory test results from Boring D3 and the four CPTs performed at the site indicate that approximately the upper 40 feet at the site are of variable densities and lack the shear strength necessary to support the anticipated structural loads for the planned parking structure without experiencing significant total and/or differential static settlements, which can potentially result in structural damage. The relatively denser/stiffer soils underlying the lower-strength soils, or an improved subgrade, are considered capable of supporting the anticipated structural loads associated with the planned parking structure.

Deep Foundation and Ground Improvement Alternatives for the Parking Structure

Deep Foundation Systems

Deep foundations such as auger cast-in-place (ACIP) or driven, precast concrete piles can provide increased support capacity and reduce total and/or differential settlements for structures with relatively high structural loads such as those anticipated for the planned parking structure. Based on the planned construction for the parking structure and the subsurface soil and groundwater conditions revealed by the explorations performed at the site, a deep foundation system consisting ACIP piles, specifically auger pressure grouted displacement (APGD) piles, or driven, precast concrete piles, extending to the deeper, competent, relatively denser/stiffer soils encountered at the site at an elevation of about -29 feet NGVD 29 (or about 40 feet below existing site grades) are feasible deep foundation alternatives to support the planned parking structure. Our experience with similar structures supported on ACIP/APGD piles indicates that supporting the parking structure on a properly designed and constructed deep foundation system extending to the relatively denser/stiffer soils referenced above can expect total static settlements on the order of about one-inch and differential settlements on the order of about ½-inch across 50 feet, or the shortest dimension of the structure, whichever is less.

Due to the noise, vibrations, and potential impact to existing development in close proximity to the site, it is our opinion the driven, precast concrete piles may not be as feasible/practical as the APGD piles for support of the planned parking structure. We have provided recommendations, including anticipated capacities, for APGD piles in this report. If the design team decides a driven pile system is a more feasible/practical deep foundation alternative for support of the parking structure, upon request we can provide supplemental recommendations regarding a driven, precast concrete pile system.

Ground Improvement Systems

An improved subgrade consisting of a properly designed and constructed Compacted Aggregate Pier (CAP) system can provide increased support capacity and reduce total and/or differential settlements for structures with relatively heavy structural loads such as those anticipated for the planned parking structure. Based on the proposed construction for parking structure and the subsurface soil and groundwater conditions revealed by the explorations performed at the site, in our opinion supporting the parking structure on conventional shallow foundations (continuous and/or isolated spread foundations, or a mat-slab foundation) supported on an improved subgrade is feasible.

Our experience with similar structures supported on similar ground improvement technology (CAP system) indicates that supporting the parking structure on a properly constructed CAP system can expect total static settlements on the order of about one-inch and differential settlements on the order of about ½-inch across 50 feet, or the shortest dimension of the structure, whichever is less.

We have provided recommendations and geotechnical parameters for design and construction of conventional shallow foundations supported over an improved subgrade consisting of a CAP system.

Soil Expansion Potential

Laboratory testing of two bulk samples consisting of representative surface near-surface clay soil collected from Borings D4 and D7 revealed these soils possess high plasticity (Plasticity Index of 34 and 40, respectively) when tested in accordance with the American Society of Testing and Materials (ASTM) D4318 test method (see Figure A2). Laboratory test results of the two bulk samples also revealed the clay soils possess an Expansion Index of 123 and 121, respectively, equivalent to a "high" expansion potential when tested in accordance with the ASTM D4829 test method (see Figures A3 and A4). Based on these laboratory test results, the near-surface clay soils at the site can significantly shrink and swell based on fluctuating moisture contents. These soils are considered to have the potential to cause significant vertical movements of shallow conventional foundations, interior floor slabs (including post-tensioned foundation slabs), exterior flatwork, and pavements.

We understand that PT slab foundations are being considered for support of the apartment and clubhouse buildings. Please note that PT slab foundations are designed to resist damage to the foundation from shrinking and swelling of expansive clay soils. However, interior and exterior wall finishes are not tolerant of the movements expected from the local clay soils. In addition, movement of the local clay soils also can adversely affect the slope and grade of exterior flatwork.

To mitigate the significant shrinking and swelling potential of the surface and near-surface clay soil, 12 to 18 inches of imported, compactable, very low-expansive (Expansion Index ≤ 20) granular soils will be required beneath interior floor slabs (including PT foundation slabs) and exterior concrete slabs-on-grade (including sidewalks and pool deck slabs). Alternatively, chemical amendment of on-site or approved imported clay soils (i.e., lime-treatment) also could be considered to reduce the shrinking and swelling potential of on-site or imported clays. Specific recommendations for subgrade preparation and engineered fill construction have been presented in this report to mitigate the effect of expansive clay soils on the planned structures and concrete slabs.

Pavement Subgrade Quality

A representative bulk sample of surface and near-surface clay soils collected from the site was subjected to Resistance ("R") value testing in accordance with California Test 301. Laboratory testing of the sample revealed the surface and near-surface clay soils possess an R-value of five (see Figure A5). Based on the lab testing results, the surface and near-surface clay soils present at the site are considered poor quality materials for support of asphalt concrete pavements and will require relatively thick pavement section to compensate for the poor quality pavement support characteristics. It is our opinion that imported soils consisting of locally derived clay, if required, also would possess the poor quality pavement support characteristics described above. Based on the assumption that pavements will be supported on near-surface clay soil present at the site or imported soils potentially consisting of clay, it is our opinion an R-value of five is appropriate for design of pavements at the site supported on untreated subgrades.

Our experience in the local area suggests that lime treatment of clay soils can result in a substantial improvement to the support characteristics of the clay soils and reduce the thickness of the required aggregate base material for pavement sections. A representative bulk sample of surface and near-surface clay soil present at the site was mixed with about four percent dolomitic quicklime and subjected to an R-value test. Laboratory test results revealed the treated clay soil possess an R-value of 87 when tested in accordance with California Test 301 (see Figure A5). Based on the Caltrans Highway Design Manual, a maximum R-value of 40 should be used for design of pavements to be supported on a treated subgrade. Therefore, an R-value of 40 is appropriate for design of pavements at the site supported on treated near-surface clay soils. Additional recommendations regarding lime-treatment of the pavement subgrade soils are provided in the Pavement Design section of this report.

Groundwater Effect on Development

Based on the borings performed at the site and review of historical groundwater data relevant to the site, we have assumed the historical high groundwater at the site to be about two feet below lowest portion of the site or about +13 feet NGVD 29. Groundwater levels at the site should be expected to fluctuate throughout the year based on variations in seasonal precipitation, time of year, local irrigation practices, the water levels of the nearby Green Valley Creek, etc.

Excavations extending below an elevation of about +13 feet NGVD 29 could encounter groundwater and require temporary dewatering during construction. For planning purposes, groundwater should be anticipated at an elevation of about +13 feet NGVD 29. Groundwater monitoring wells could be installed in different areas of the site prior to construction to evaluate actual groundwater levels before and during construction. If groundwater is encountered, the use of sumps, submersible pumps, deep wells or a well point system could be used as methods to lower the groundwater level at least two feet below the bottom of excavations. The dewatering method used will depend on the soil conditions, depth of the excavation and amount of groundwater present within the excavation. Dewatering, if required, should be the dewatering contractor's responsibility. The dewatering system should be designed and constructed by a dewatering contractor with local experience.

Due to the potential of a relatively high groundwater elevation at the site, we have included specific recommendations in this report regarding the construction and design of the planned swimming pool to account for hydrostatic pressures.

<u>Seasonal Water</u>

During the wet season, infiltrating surface runoff water will create a saturated surface condition of the near-surface soils. It is probable that grading operations attempted following the onset of winter rains and prior to prolonged drying periods will be hampered by high soil moisture contents. Such soil, intended for use as engineered fill, will require a prolonged period of dry weather and/or considerable aeration to reach a moisture content that allows achieving the required compaction. This should be considered in the construction schedule for the project.

<u>Excavation Conditions</u>

The surface and near-surface soils at the site should be readily excavatable with conventional earthmoving and trenching equipment. Based on the explorations performed at the site and review of historical groundwater data relevant to the site, excavations extending below an elevation of about +13 feet NGVD 29 could encounter groundwater and require dewatering (depending on the time of year). Therefore, excavations associated with trenches for underground utilities, excavations associated with the construction of the swimming pool, and any other excavations associated with the planned construction extending below an elevation of about +13 feet NGVD 29 could require temporary dewatering during construction. Please refer to the <u>Groundwater Effect on Development</u> section in this report for our discussion of dewatering alternatives.

After excavations have been properly dewatered (if required), the soils within the excavation sidewalls will remain in a saturated condition and potentially create unstable conditions that can result in caving or sloughing. The presence of cohesionless or disturbed soils may also create unstable conditions that can also result in caving or sloughing. If any of these conditions exist, the contractor should be prepared to brace or shore these shallow excavations as needed. Excavations left open for more than a day also may be susceptible to caving or sloughing; therefore, such excavations should be evaluated by the contractor on a daily basis and determine if it is necessary to brace or shore the excavations. Bracing or shoring of any excavations, if necessary, should conform to current Occupational Safety and Health Administration (OSHA) requirements.

Excavations or trenches exceeding five feet in depth that will be entered by workers should be sloped, braced or shored to conform to current OSHA requirements. The contractor must provide an adequately constructed and braced shoring system in accordance with federal, state and local safety regulations for individuals working in an excavation that may expose them to the danger of moving ground.

Temporarily sloped excavations should be constructed no steeper than a one-and-a-half horizontal to one vertical (1½H:1V) inclination. Temporary slopes likely will stand at this inclination for the short-term duration of construction, provided significant pockets of loose and/or saturated granular soils are not encountered. Flatter slopes would be required if these conditions are encountered.

Excavated materials should not be stockpiled directly adjacent to an open excavation to prevent surcharge loading of the excavation sidewalls. Excessive truck and equipment traffic should be avoided near excavations. If material is stored or heavy equipment is stationed and/or operated near an excavation, a shoring system must be designed to resist the additional pressure due to the superimposed loads.

On-site Soil Suitability for Use in Fill Construction

From a geotechnical standpoint, the on-site soils are considered suitable for use as engineered fill provided that they do not contain significant quantities of organics, rubble and deleterious debris, and are at a proper moisture content to achieve the desired degree of compaction. Organically laden topsoil should not be reused as engineered fill.

However, clay soils present beneath the site are not suitable for direct support of interior or exterior concrete slabs-on-grade (including swimming pool deck slabs), unless they are lime-treated. Specific recommendations for subgrade preparation and engineered fill construction have been presented in this report to mitigate the effect of expansive clay soils on the planned structures and concrete slabs.

Soil Corrosion Potential

Three representative samples of surface and near-surface clay soil present at the site were submitted to Sunland Analytical of Rancho Cordova, California for testing to determine minimum resistivity, pH, and chloride and sulfate concentrations to help evaluate the potential for corrosive attack upon reinforced concrete and buried metal. The results of the corrosivity testing are summarized in Table 2. Copies of the corrosion test reports are presented in Figures A6 through A9.

Table 2: Corrosion Test Results

Analyte	Test Method	Sample: D4 (0 – 3')	Sample: D7 (0 – 3')	Sample: D3-4I (16' – 16½')
pH	CA DOT 643 Modified*	6.84	6.89	7.19
Minimum Resistivity	CA DOT 643 Modified*	640 Ω-cm	380 Ω-cm	670 Ω-cm
Chloride	CA DOT 422	100.3 ppm	77.4 ppm	7.8 ppm
Sulfate	CA DOT 417	181.1 ppm	411.0 ppm	33.6 ppm
	ASTM D516	190.5 mg/kg	427.0 mg/kg	36.5 mg/kg

Notes: * = Small cell method; Ω-cm = Ohm-centimeters; ppm = parts per million; mg/kg = milligrams per kilogram

The California Department of Transportation Division of Engineering Services Materials Engineering and Testing Services Corrosion Branch, *Corrosion Guidelines*, Version 3.0, dated March 2018, considers a site to be corrosive to structural elements if one or more of the following conditions exists for representative soil and/or water samples taken: has a chloride concentration greater than or equal to 500 ppm, sulfate concentration greater than or equal to 1,500 ppm, or the pH is 5.5 or less. Based on this criterion, the on-site soils tested are not considered corrosive to steel reinforcement properly embedded within Portland cement concrete (PCC). However, the relatively low resistivity test results of the samples tested indicate the surface and near-surface clay soils may be moderately to highly corrosive to unprotected metal.

Table 19.3.1.1 – *Exposure Categories and Classes* of American Concrete Institute (ACI) 318-19, Section 19.3 – Concrete Durability Requirements, as referenced in Section 1904.1 of the *2019 CBC*, indicates the severity of sulfate exposure for the samples tested from Borings D4 and D7 is Exposure Class *S1*. Exposure Class *S1* is assigned for structural concrete members in direct contact with soluble sulfates in soil or water. Table 19.3.2.1 – *Requirements for Concrete by Exposure Area* of ACI 318-19 lists the appropriate types of cement (Type II), the maximum water-cementitious material ratio (maximum w/cm = 0.5), and the minimum specified compressive strength of concrete (minimum f_c' = 4,000 psi) for Exposure Class *S1*. For Exposure *S1*, ASTM C150 Type II cement is limited to a maximum tricalcium aluminate (C_3A) content of 8.0 percent, assuming a minimum concrete cover as detailed in ACI 318-19 is maintained for all reinforcement. Blended cements under ASTM C595 with MS (moderate sulfate resistance) designation also are appropriate for use. Under ASTM C1157, the appropriate designation for Exposure Class *S1* is Type MS.

Wallace-Kuhl & Associates are not corrosion engineers. Due to the corrosive test results for the soil samples tested from the site, we recommend that a corrosion engineer is consulted to further evaluate the corrosion potential of the on-site soils and provide specific corrosion mitigation measures that are appropriate for the proposed development.

Environmental Concerns

Review of the *Phase I Environmental Site Assessment* (WKA No. 13081.01) revealed the following conclusions:

- According to an environmental lien search, no environmental liens are associated with the site.
- Given the documentation reviewed concerning the agency listings for neighboring facilities, none of the facilities reviewed are likely to have a negative impact on the site.
- Based on the completion of a vapor encroachment condition (VEC) screening matrix, it was concluded a VEC can be ruled out for the site because a VEC does not or is not likely to exist.
- No further environmental assessment is warranted at this time.

For additional information regarding environmental concerns at the site please refer to the *Phase I Environmental Site Assessment* report.

RECOMMENDATIONS

General

The recommendations presented below are appropriate for typical construction in the late spring through fall months. The on-site soils likely will be saturated by rainfall in the winter and early spring months, and will not be compactable without drying by aeration, chemical treatment, or geogrid stabilization. Should the construction schedule require work to begin during the wet months, additional recommendations can be provided, as conditions dictate.

Site preparation should be accomplished in accordance with the provisions of this report and the appended guide specifications. A representative of the Geotechnical Engineer should be present during all earthwork operations to evaluate compliance with our recommendations and the guide specifications included in this report. The Geotechnical Engineer of Record referenced herein is considered the Geotechnical Engineer that is retained to provide geotechnical engineering observation and testing services during construction.

Site Clearing

Prior to grading, the construction areas should be cleared of all surface trash, rubble, and deleterious debris to expose firm and stable soils, as determined by the Geotechnical Engineer's representative. The area of removal should extend at least five feet beyond the edge of all exterior foundations and also at least five feet beyond any exterior flatwork or pavements, where practical. Any rubble and debris should be removed from the site.

Any existing underground utilities designated to be removed or relocated should include removal of all trench backfill and associated granular bedding material. The resulting excavations should be replaced with engineered fill. On-site wells, septic systems, or tanks were not noted during our field exploration or document review; however, if any of these items are discovered, they should be properly abandoned in accordance with Solano County requirements.

Existing surface vegetation and organically laden soil within construction areas should be removed by stripping. Debris from the stripping should not be used in general fill construction areas supporting the planned buildings, concrete slabs or pavements. With prior approval from the Geotechnical Engineer, strippings may be used in landscape areas, provided they are kept at least five feet from the building pads, pavements, concrete slabs and other surface improvements, moisture conditioned, and compacted.

Discing of the organics into the surface soils may be a suitable alternative to stripping, depending on the condition and quantity of the organics at the time of grading. The decision to utilize discing in lieu of stripping should be made by the Geotechnical Engineer, or his representative, at the time of earthwork construction. Discing operations, if approved, should be observed by the Geotechnical Engineer's representative, and be continuous until the organics are adequately mixed into the surface soils to provide a compactable mixture of soil containing minor amounts of organic matter. Pockets or concentrations of organics will not be allowed.

Any trees, bushes, or other vegetation designated for removal should include the entire rootball and roots larger than ½-inch in diameter. Adequate removal of debris and roots may require laborers and handpicking to clear the subgrade soils to the satisfaction of the Geotechnical Engineer's on-site representative.

Depressions resulting from site clearing operations, as well as any loose, soft, disturbed, saturated, or organically contaminated soils, as identified by the Geotechnical Engineer's representative, should be cleaned out to firm, undisturbed soils and backfilled with engineered fill in accordance with the recommendations of this report. It is important that the Geotechnical Engineer's representative be present during clearing operations to verify adequate removal of the surface and subsurface items, as well as the proper backfilling of resulting excavations.

Subgrade Preparation

Approximately the upper one to two feet of soils across the site, including the previously constructed building pads, are in a relatively soft condition, likely due to the surface and near-surface soils at the site being subjected to several years of seasonal drying and wetting. In our opinion, these soils should not be relied upon to support structural improvements associated with the apartment complex in their current condition. Therefore, following site clearing activities all building pad areas should be over-excavated to a depth of at least 12 inches below the lowest existing ground surface elevation within the building pad footprint or at least 12 inches below the final soil subgrade elevation, whichever is deeper. The over-excavation should extend at least five feet beyond the edge of exterior foundations or the building footprint, whichever is greater. The intent of these recommendation is to construct uniform building pads, provide uniform support for the planned buildings, and reduce the potential of differential settlements. We recommend the extents of the required over-excavation be clearly marked on the final civil engineering or grading plans. Any debris exposed by the required over-excavation should be removed. *This over-excavation recommendation is not necessary within areas designated as exterior flatwork or pavements outside of the parking structure, except as necessary to remove construction debris or disturbed soil where encountered*.

After over-excavation operations have been performed, the Geotechnical Engineer's representative should evaluate the exposed subgrade soils to determine if additional over-excavation is required due to disturbed subgrade soils or if watering of the exposed subgrade is required to mitigate potential desiccation cracks deeper than 12 inches below the exposed subgrade. If desiccation cracks within the exposed subgrade are less than 12 inches deep, the exposed subgrade soils, as well as any other surfaces to receive fill, achieved by excavation or remain at grade, should be scarified to a depth of at least 12 inches, thoroughly moisture conditioned to at least two percent above the optimum moisture content, and uniformly compacted to at least 90 percent relative compaction. Relative compaction should be based on the maximum dry density as determined in accordance with the ASTM D1557 Test Method. Due to the properties of the highly plastic/expansive surface and near-surface soils present at the site, consideration should be given to using rotary mixing equipment to thoroughly process the upper 12 in of structural areas (as described above).

It is possible that soils present at the bottom of required excavations initially will be too wet to properly compact and will require a period of drying and/or considerable aeration for the soils to dry to a workable moisture content. Alternative recommendations to stabilize the bottom of excavations can be provided upon request based on actual field conditions. The use of lime stabilization or use of geotextile fabrics or geogrids is typically recommended to stabilize soils during construction.

Compaction of soil subgrades should be performed using a heavy, self-propelled, sheep's-foot compactor capable of achieving the required compaction and must be performed in the presence of the Geotechnical Engineer's representative who will evaluate the performance of subgrade under compactive load. Difficulty in achieving subgrade compaction may be an indication of loose, soft or unstable soil conditions associated with prior site development and/or activities. If these conditions exist, the loose, soft or unstable materials should be excavated to expose firm and stable soils. The resulting excavations should be backfilled with engineered fill compacted in accordance with the recommendations in this report.

Engineered Fill Construction

To mitigate the significant shrinking and swelling potential of the surface and near-surface clay soils present at the site, at least 12 inches of imported, compactable, very low-expansive (Expansion Index ≤ 20) granular soils will be required beneath all interior floor slabs (including PT foundation slab) and exterior flatwork. Alternatively, chemical amendment of on-site clay soils (i.e. lime-treatment) could also be considered to reduce the shrinking and swelling potential of on-site or imported clays. If the lime-treatment alternative is selected, the upper 12 inches of final soils subgrades for all interior floor slabs and exterior flatwork should be constructed as described in the Lime Treatment Alternative section of this report. The thickness of the imported, compactable, very low-expansive granular soils or lime-treated clay soils should be increased to 18 inches beneath swimming pool deck slabs. Interior and exterior concrete slab-on-grade final soil subgrade is defined as the surface in which aggregate base or capillary break materials are placed.

The construction of the very low-expansive layer described above is not required for concrete slabs-on-grade designated for vehicle support such as those for the planned parking structure. Concrete slabs-on-grade designated for vehicle support should be designed in accordance with the recommendations provided in the Pavement Design section of this report.

On-site Soils

From a geotechnical standpoint, the on-site soils encountered in our explorations are considered suitable for use as engineered fill, provided they are at a workable moisture content to achieve required compaction, and do not contain rubbish, rubble, deleterious debris, and organics. However, due to their significant expansion potential, on-site clay soils should not be used in fills within the upper 12 inches of final soil subgrade beneath interior floor-slabs and exterior flatwork, as defined above, or within the upper 18 inches of final soil subgrade beneath swimming pool deck slabs, unless the clay soils are lime-treated as described in the Lime Treatment Alternative section of this report.

Imported Fill Materials

The surface and near-surface soils at the site do not meet the very low-expansive criteria; therefore, we assume imported fill materials that meet the very low-expansive criteria would be required for the upper 12 inches of soil subgrade beneath interior floor-slabs and exterior flatwork, as defined above, and for the upper 18 inches of soil subgrade beneath swimming pool deck slabs. Imported fill materials to be used in the upper 12 inches of soil subgrade beneath interior floor slab and exterior flatwork, as defined above, and in the upper 18 inches of soil subgrade beneath the swimming pool deck slabs, should be compactable, well-graded, granular soils with a Plasticity Index of 15 or less when tested in accordance with ASTM D4318; an Expansion Index of 20 or less when tested in accordance with ASTM D4829, and should not contain particles greater than three inches in maximum dimension.

Imported fill materials to be used within pavement areas, below an elevation of 12 inches of the final soil subgrade beneath interior floor slabs or exterior flatwork, below an elevation of 18 inches of the final soil subgrade beneath swimming pool deck slabs, or to be lime-treated and used beneath interior floor-slabs, exterior flatwork, swimming pool deck slabs, or pavements can consist of locally derived clayey soils provided they are compactable, possess a Plasticity Index of 40 or less and a Liquid Limit of 64 or less when tested in accordance with ASTM D4318; and possess an Expansion Index of 123 or less when tested in accordance with ASTM D4829. The clay material should not contain particles greater than three inches in maximum dimension, should be free of significant organics, and should be at a moisture content that allows the desired degree of compaction.

In addition, with the exception of imported aggregate base and bedding/initial fill materials for underground utilities, we recommend that the contractor provide appropriate documentation for all imported fill materials that designates the import materials do not contain known contaminants per Department of Toxic Substances Control's guidelines for clean fill, and have corrosion characteristics within acceptable limits.

Imported soils must be approved by the Geotechnical Engineer prior to being transported to the site.

General

Engineered fill consisting of on-site or import materials should be placed in lifts not exceeding six inches in compacted thickness, with each lift being thoroughly moisture conditioned to at least two percent above the optimum moisture content for clay soils and to the optimum moisture content for granular soils (sandy/silty soils), maintained in that condition, and uniformly compacted to at least 90 percent relative compaction. Fill materials within the within the footprint of buildings that are deeper than five feet below the final soil subgrade elevation, if any, should be compacted to at least 92 percent relative compaction, at a moisture content of at least the optimum moisture content for granular soils and at least two percent above the optimum moisture content for clay soils.

The upper six inches of untreated pavement subgrades should be uniformly compacted to at least 95 percent relative compaction at a moisture content of at least two percent above the optimum moisture content, regardless of whether final grade is established by excavation, engineered fill or left at grade. If pavement subgrades will be lime-treated, the upper 12 inches of lime-treated subgrade soils should be compacted to at least 95 percent relative compaction at not less than two percent over the optimum moisture content. Additional recommendations regarding lime-treatment of the pavement subgrade soils are provided in the Pavement Design section of this report.

Subgrades for support of all concrete slabs-on-grade and pavements should be protected from disturbance or desiccation until covered by capillary break material or aggregate base. Disturbed or desiccated subgrade soils may require additional processing and recompaction, depending on the level of disturbance.

Permanent excavation and fill slopes should be constructed no steeper than two horizontal to one vertical (2:1) and should be vegetated as soon as practical following grading to minimize erosion. As a minimum, the following erosion control measures should be considered: placement of straw bale sediment barriers or construction of silt filter fences in areas where surface run-off may be concentrated. Slopes should be over-built and cutback to design grades and inclinations.

All earthwork operations should be accomplished in accordance with the recommendations contained within this report and the guide specifications included in this report. We recommend the Geotechnical Engineer's representative be present on a regular basis during all earthwork operations to observe and test the engineered fill and to verify compliance with the recommendations of this report and the project plans and specifications.

Lime-treatment Alternative

As an alternative to the use of imported, very low-expansive (Expansion Index ≤ 20), granular soils beneath interior and exterior concrete slabs-on-grade, amendment of the on-site or approved imported clay soils with lime is expected to mitigate the effect of expansion pressures on interior and exterior concrete slabs-on-grade produced by untreated clay soils. Based on our experience in the local area, the clay soils encountered at the site and imported clay soils that meet the clay soil criteria described in the Engineered Fill Construction section of this report are anticipated to react well with the addition of quicklime (high-calcium or dolomitic). If lime-treatment of on-site or approved imported clay soils is selected for the final soil subgrade beneath interior and exterior concrete slabs-on-grade, we recommend the final soil subgrade elevation beneath interior concrete slabs and exterior flatwork is mixed with lime at a minimum spread rate of at least 4½ pounds of quicklime per square foot of treated soil, at a depth sufficient to produce a compacted lime-treated layer 12 inches thick. For swimming pool deck slabs, we recommend the final soil subgrade beneath the slabs is mixed with lime at a minimum spread rate of at least 6¾ pound of quicklime per square foot of treated soils, at depth sufficient to produce a compacted lime-treated layer 18 inches thick. Lime should be mixed into the soil, allowed to cure for a period of 12 to 72 hours, remixed, moisture conditioned and compacted.

Lime-treatment of clay subgrade soils should be performed in general conformance with Section 24 of the *Caltrans Standard Specifications*, latest edition. Lime-treated soil beneath interior and exterior concrete slabs-on-grade should be compacted to at least 90 percent relative compaction at no less than two percent over the optimum moisture content and maintained in that condition until covered by capillary break gravel or aggregate base. Treatment of more than 12 inches of soil may require the use of a mixing table and compaction of the treated soils in lifts. The contractor must use equipment that will provide uniform and complete compaction of the entire lime-treated section.

Utility Trench Backfill

Utility trench backfill should be mechanically compacted as engineered fill in accordance with the following recommendations. Bedding of utilities and initial backfill around and over the pipe should conform to the manufacturer’s recommendations for the pipe materials selected and applicable sections of the governing agency standards. If open-graded, crushed rock is used as bedding or initial backfill, an approved geotextile filter fabric should be used to separate the material from overlying finer-grained soils. Due to the relatively shallow groundwater elevation at the site, if the crushed rock extends deeper than an elevation of about +13 feet NGVD 29, the crushed rock material should be fully enveloped with the filter fabric. The intent of these recommendations is to reduce the potential of migration of fine-grained soils into the crushed rock (soil piping), which can result in settlement.

We recommend that on-site soil or approved import material be used as trench backfill. Utility trench backfill should be placed in relatively thin lifts, thoroughly moisture conditioned to at least the optimum moisture content for granular soils (sand or silt) and two percent above the optimum moisture content for clay soils, and mechanically compacted to at least 90 percent relative compaction. The lift thickness will be dependent on the type of compaction equipment used.
Within the final subgrade thickness recommended beneath interior and exterior concrete slabs-on-grade (12 to 18 inches), trench backfill should consist of imported very low-expansive, granular material as described in the Engineered Fill Construction section of this report, unless the lime-treatment alternative is selected. If the upper 12 to 18 inches of final subgrade beneath interior and exterior concrete slabs-on-grade (including swimming pool deck slabs) consist of lime-treated clay soils, the upper 12 to 18 inches of trench backfill should consist of approved controlled density fill or aggregate base compacted to at least 95 percent relative compaction. Within the upper six inches of untreated pavement subgrade soils and upper 12 inches of lime-treated pavement subgrade soils, compaction should be increased to at least 95 percent relative compaction at no less than two percent above the optimum moisture content.

Excavations extending below an elevation of about +13 feet NGVD 29 could encounter groundwater and require temporary dewatering depending on the time of year. Please refer to the Groundwater Effect on Development section in this report for our discussion of dewatering alternatives. Regardless, based on the relatively high in-place moisture content of the near-surface soils, it is likely that materials excavated from trenches will be at elevated moisture contents and will require significant aeration or a period of drying to reach a compactable moisture content. We recommend bid documents contain a unit price for the removal and drying of saturated soils, or replacement with approved import soils.

We recommend that all underground utility trenches aligned nearly parallel with new foundations be at least three feet from the outer edge of foundations, wherever possible. As a general rule, trenches should not encroach into the zone extending outward at a one horizontal to one vertical (1H:1V) inclination below the bottom of foundations. Additionally, trenches parallel to existing foundations should not remain open longer than 72 hours. The intent of these recommendations is to prevent loss of both lateral and vertical support of foundations, resulting in possible settlement.

Conventional Shallow Foundation Design for the Apartment and Clubhouse Buildings

The planned apartment and clubhouse buildings may be supported on a PT slab foundation system or conventional shallow foundations with interior slab-on-grade lower floors. Recommendations for PT slab foundations and conventional foundations are provided below.

Post-Tensioned Slab Foundations

Design of PT slab foundation systems should be performed by a qualified Structural Engineer using the geotechnical engineering parameters provided in Table 3, which were derived from guidelines contained in the Post-Tensioning Institute manual, *Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive Soils (PTI DC10.5-2012).*

Table 3: Post-tensioned Slab Design Parameters

1.	Thornthwaite Moisture Index = **-20**
2.	Average Edge Moisture Variation Distance (e_m): Center Lift = **5½ feet** Edge Lift = **3 feet**
3.	Expansion Index = **123**
4.	Plasticity Index = **34**
5.	Plastic Limit = **29**
6.	Liquid Limit = **63**
7.	Percent Fine Clay = **55%** (≤ 0.002 mm/≤ 0.075 mm)
8.	Activity Ratio (Ac) = **0.62** (Plasticity Index/Percent Fine Clay)
9.	Zone = **III**
10.	Approximate Depth to Constant Moisture = **5.0 feet**
11.	Approximate Soil Suction = **3.9 pF**
12.	Anticipated Swell (y_m): Center Lift = **¾-inch** Edge Lift = **2 inches**

The PT slab foundation should not exert more than 1,500 pounds per square foot (psf) on the building pad soils for the dead plus live load conditions. The allowable PT slab bearing capacity may be increased by one-third for total load, including wind or seismic forces.

The project Structural Engineer should determine the appropriate thickness of the PT slab foundations; however, a minimum 10-inch thick slab, deepened to 12 inches at the perimeter, is typically used in the local area. Temporary loads exerted during construction from vehicle traffic, construction equipment, storage of palletized construction materials, etc. also should be considered in the design of the thickness and reinforcement of the PT slab foundation.

We recommend the interior portion of the PT slab (not the deepened permitter) be underlain by a durable vapor retarder (at least 10 mils thick) placed directly on the soil subgrade. The plastic water vapor retarder should meet or exceed the standard specifications described in ASTM E1745, and be installed in strict conformance with the manufacturer's guidelines. The plastic water vapor retarder may be covered with about two inches of damp, clean sand, or pea gravel. Prior to placement of the vapor barrier, at least the upper 12 inches of final soil subgrade should be brought to a moist condition and maintained in that condition. If the building pads become dry and desiccated, the building pads will require processing and compaction prior to foundation construction. The Geotechnical Engineer's representative should confirm the subgrade soils are at the appropriate moisture content within 72 hours of slab construction, prior to placement of the vapor retarder membrane.

Conventional Shallow Foundations

The planned apartment and clubhouse buildings could also be supported upon a continuous perimeter foundation with continuous and/or isolated interior spread foundations embedded at least 18 inches below lowest adjacent soil grade, provided the subgrade has been prepared in accordance with the recommendations included in this report. Lowest soil grade is defined as either the adjacent exterior soil grade or the soil subgrade beneath the building, whichever is lower. A continuous, reinforced foundation should be utilized for the perimeter of the buildings to act as a "cut-off" to help minimize moisture infiltration and variations beneath the interior slab-on-grade areas of the buildings. Continuous foundations should maintain a minimum width of 18 inches and isolated spread foundations should be at least 24 inches in plan dimension.

Foundations may be sized for maximum allowable "net" soil bearing pressures of 2,500 pounds per square foot (psf) for dead plus live loads, with a 1/3 increase to evaluate total loads including the short-term effects of wind or seismic forces. The weight of the foundation concrete extending below lowest adjacent soil grade may be disregarded in sizing computations.

We recommend that all foundations be adequately reinforced to provide structural continuity, mitigate cracking and permit spanning of local soil irregularities. The project Structural Engineer should determine final foundation reinforcement.

Lateral Resistance

Resistance to lateral foundation displacement for PT slabs and conventional shallow foundations may be computed using an ultimate friction factor of 0.30, which may be multiplied by the effective vertical load on each foundation. Additional lateral resistance may be computed using an ultimate passive earth pressure equivalent to a fluid pressure of 300 psf per foot of depth, acting against the vertical projection of the foundation.

These two modes of resistance should not be added unless the frictional component is reduced by 50 percent since full mobilization of the passive resistance requires some horizontal movement, effectively reducing the frictional resistance.

Ground Improvement and Deep/Pile Foundation Design Alternatives for the Parking Structure

Based on the anticipated structural loads for the planned parking structure and the subsurface soil and groundwater conditions encountered at the site, in our opinion both a shallow foundation system (e.g. continuous and/or isolated spread footings or a mat foundation) supported on an improved subgrade consisting of a properly designed and constructed CAP system and APGD piles and are suitable alternatives for support of the parking structure. Therefore, preliminary recommendations for shallow foundation supported on a CAP improved subgrade and recommendations for APGD piles and are provided below. Alternative foundation may be considered at the site and can be evaluated upon request.

Conventional Shallow Foundations on a CAP System

We anticipate the planned parking structure could be supported on continuous and/or isolated spread foundation, or a mat-slab foundation, supported on a CAP system. A CAP system consists of drilled shafts backfilled with compacted aggregate base and is considered capable of densifying the subsurface soils at the site to provide adequate support for the planned parking structure. This would result in an increase in the ultimate bearing capacity and mitigation of some of the static settlements. A qualified CAP contractor licensed in the State of California should be contacted directly to provide final design recommendations for the CAP system, including shaft lengths, allowable capacities and post-construction, static total and differential settlements.

The depth of the CAP system will depend on the amount of static settlements that can be tolerated by the planned parking structure from a structural and architectural standpoint. For preliminary purposes, we have assumed that extending the CAP system to at least 20 feet below the final soil subgrade elevation would result in total static settlements on the order of about one-inch and differential settlements on the order of about ½-inch across 50 feet, or the shortest dimension of the structure, whichever is less.

Continuous and/or isolated spread foundations bearing on a CAP improved subgrade should extend at least 18 inches below the lowest adjacent soil grade, provided the subgrade has been prepared in accordance with recommendations included in this report. Continuous foundation should maintain a minimum width of 18 inches and isolated spread foundations should be at least 24 inches in plan dimension.

A mat foundation bearing on a CAP improved subgrade should extend at least 18 inches below the lowest adjacent soils, provided the subgrade has been prepared in accordance with recommendations included in this report.

The allowable bearing capacity of conventional shallow foundations constructed over a CAP system would be on the order of 5,000 psf for dead plus live load condition, assuming a properly installed CAP system. The deflection of a mat foundation can be evaluated using an allowable modulus of subgrade reaction (ks) of 150 psi. The CAP system layout, shaft length, final bearing pressures, cell capacities and actual settlement will depend on the actual loading conditions for the parking structure and should be determined by the CAP designer. The final bearing pressures and cell capacities should include an appropriate factor of safety. The weight of foundation concrete extending below adjacent soil grade may be disregarded in sizing computations.

All foundations should be adequately reinforced to provide structural continuity, mitigate cracking and permit spanning of local soil irregularities. The structural engineer should determine final foundation reinforcement. For the mat foundation, the structural engineer or slab designer should determine the mat thickness and design the mat to transmit the loads associated with the parking structure uniformly across the entire slab.

Resistance to lateral foundation displacement for conventional shallow foundations supported on a CAP system may be computed using an allowable friction factor of 0.30 for soil subgrade and 0.40 for aggregate base (CAPs), which may be multiplied by the effective vertical load on the foundation. Additional lateral resistance may be computed using an allowable passive earth pressure of 300 psf per foot of depth, acting against vertical projection of the foundations. These two modes of resistance should not be added unless the frictional value is reduced by 50 percent since full mobilization of these resistances typically occurs at different degrees of horizontal movement, effectively reducing the frictional resistance.

Auger Pressure Grout Displacement (APGD) Piles

We anticipate the planned parking structure also can be supported on a deep foundation system consisting of APGD piles. APGD piles should be a minimum of 18 inches in diameter and extend to a minimum tip elevation of about -21 feet NGVD 29, which correlates to a depth of about 40 feet below existing site grades.

APGD piles may be designed utilizing the following maximum allowable loads per pile with appropriate factor of safety (F.S.) as summarized in Table 4. Alternate pile capacities for piles of different dimensions can be provided upon request. The factors of safety used to determine the allowable pile capacities presented below are based on our experience with similar projects and current industry standards. For design purposes, the factor of safety for the dead load condition only may be modified by the Structural Engineer if considered appropriate. APGD pile concrete should achieve a minimum compressive strength of 4,000 psi when tested in accordance with ASTM C109.

Table 4: Allowable APGD Pile Capacities

Loading Conditions		18-inch-diameter	
		Allowable Pile Capacity (kips)	Ultimate Pile Capacity (kips)
Axial Compression	DL (F.S. = 3)	100	300
	DL + LL (F.S. = 2)	150	300
	Total Load (F.S. = 1.5)	200	300
Axial Uplift (Tension)	Total Load (F.S. = 2)	75	150
Lateral Load (½-inch deflection)	Fixed Head (F.S. = 1.5)	33	50
	Free Head: (F.S. = 1.5)	16	25

Notes: DL = Dead Load; LL = Live Load; F.S. = Factor of Safety

Reductions in axial compression capacity for consideration of group action are not considered necessary, provided piles are spaced no closer (center-to-center) than three times the diameter of the pile. The indicated uplift pile capacity is based upon the assumption that the piles will be properly reinforced to transfer pullout forces to the pile cap. The weight of pile cap concrete extending below grade and the weight of each pile may be disregarded in determinations of the net compressive load transmitted to the supporting soil. Lateral resistance for pile caps may be computed using an allowable passive earth pressure equivalent to a fluid pressure of 300 psf per foot of depth, acting against the pile caps.

The allowable pile capacities for the APGD piles shown above in Table 4 are recommended with the stipulation that a pile load-testing program be performed prior to the installation of production piles. A representative of the Geotechnical Engineer must be present full time during all pile construction activities, including the construction of piles for the load-testing program, to record and document construction of each pile.

Specific lateral loading information was not available at the time this report was prepared. To assist in evaluation of the lateral reinforcement requirements for the ACIP/APGD piles, we have utilized the computer program L-PILE (Version 2013.7.07) provided by Ensoft, Inc. Soil parameters used in the L-Pile analysis are summarized below in Table 5.

Table 5: Soil Parameters for L-PILE Analysis

L-Pile Soil Type	Approximate Depth Below Existing Ground Surface (feet)	Approximate Ground Surface Elevation (feet NGVD 29)		Effective Unit Weight (pcf)	Undrained Cohesion (psf)	Friction Angle (degrees)	Soil Modulus, k-value (pci)	Strain at 50% Stress, (ε50)
		Top	Bottom					
Stiff Clay without Free Water	0 to 6	+19	+13	110	1,500	---	500	0.007
Stiff Clay with Free Water	6 to 25	+13	-6	48	1,500	---	500	0.007
Medium Dense Sand with Free Water	25 to 30	-6	-11	58	0	35	60	---
Very Stiff Clay with Free Water	30 to 35	-11	-16	53	3,000	---	1,000	0.005
Medium Dense to Dense Sand with Free Water	35 to 40	-16	-21	63	0	38	125	---

Notes: pcf = pounds per cubic foot; psf = pounds per square foot; pci = pounds per cubic inch

We modeled a single APGD pile for “free-head” and “fixed-head” conditions subjected to a pile head deflection of ½-inch under static conditions. The resulting lateral load capacities for “free-head” and “fixed-head” conditions are presented above in Table 4. Pile fixity may be determined by utilizing the deflection, shear and moment diagrams included in Appendix D.

Piles that are spaced closer than eight pile diameters from each other can be analyzed for lateral group effects by using the appropriate modification factors (P-multipliers, P_m) summarized in Table 6. The P_m multiplier values were determined for various pile spacing’s using the equations developed after extensive testing of pile groups in the western United States by Dr. Kyle Rollins (Rollins, 2003).

Table 6: Modification Factors (P-multipliers)

Row	Pile Diameter Spacing's		
	3	4	5 or more
1st (Lead) Row	0.79	0.86	0.92
2nd Row	0.57	0.72	0.84
3rd Row and higher	0.41	0.58	0.72

Pile Load Testing Program

A pile loading testing program conducted prior to installation of production piles is recommended to determine the correct length for each pile type to achieve the **ultimate capacity** of the piles. The pile load test program typically includes a "quick" static load test(s) and/or pile driving analyzer (PDA) tests. PDA testing for pre-construction piles can be used to develop a correlation between static load test results; the PDA testing can also be used during the construction of test or production piles in lieu of "quick" load tests. An advantage of PDA testing over the "quick" load pile testing is the savings in time to set up the load test frame that typically takes three to five days, and a "quick" load test program often takes about eight hours per pile to complete. All other construction activities at the site would have to be temporarily stopped during the load testing programs.

Quick Load Testing

For quick load testing, the pile load test frame and supply of the personnel and equipment necessary to conduct the load tests should be constructed in accordance with the latest version of the following test methods: ASTM D1143 for compressive loads, ASTM D3689 for tensile loads, and ASTM D3966 for lateral loads, as specified in the *Guide Specifications for Auger Cast-in-Place (ACIP) Piles* provided in Appendix F.

We recommend that three ACIP/APGD test piles be cast-in-place to reach a minimum tip depth of at least -25 feet NGVD 29, which correlates to a depth of about 40 feet below existing site grades. Additional test piles are recommended if multiple pile sizes are used in the design or if alternate pile capacities are being considered. Final tip elevations for the production ACIP/APGD piles would be determined by the Geotechnical Engineer following completion of the load testing. The reaction system should be capable of resisting forces from tests on the ACIP/APGD test piles in axial compression and tension as specified in Table 4. The test piles should be tested in compression and tension, as necessary. In addition, a lateral load test could be performed between adjacent piles. One or more of the piles may be loaded to failure in any of the test configurations.

Submittals for the load testing frame, hydraulic pumps, hydraulic jacks, dial indicators, and calibration documentation must be provided by the pile contractor in accordance with the project plans and specifications. Document submittals should be provided at least 48-hours in advance of the pile load tests.

Prior to beginning load tests, the pile concrete should achieve a minimum compressive strength of 4,000 psi when tested in accordance with ASTM C 109. Construction activities at the site must be restricted during the load-testing program. However, construction activities may proceed during the setup of the load frame and installation of the test piles. Excessive vibration of the ground near the load test can cause movement of the test frame and the sensitive pile deflection measurement devices. Using the ASTM "quick loading" method, the compression tests will run for about eight hours per pile and the tension testing will run for about four hours per pile.

Final pile construction criteria would be determined from the results of the load-testing program. We recommend that the pile load test setup be located outside the location of any permanent pile caps or grade beams, and that the test piles and reaction piles be abandoned upon completion of the testing.

Pile Driving Analyzer Testing

PDA testing involves instrumentation of the piles and recording the response of the pile during dynamic loading. PDA testing consists of dropping a heavy weight from a certain height on to the pile head and monitoring the response of the pile. The capacity of the pile can be computed from the analyses of the PDA test. ACIP/APGD piles subjected to quick load testing can also be subjected to PDA testing, provided the piles are not damaged during the quick load tests. As an alternative, PDA testing can also be performed on different piles designated for load testing (non-production piles), provided the piles extend to the minimum tip elevation described above.

Additional PDA testing should be performed during construction of production ACIP/APGD piles in the event that as-built pile dimensions differ from the recommended dimensions, which could result from refusal to auger penetration or in random areas across the site to verify that the earth materials are supporting the piles as indicated by the load test program.

Please note there is some risk of damage to piles from static and dynamic load testing. Any production piles that are damaged during testing should be evaluated by the Geotechnical Engineer, Structural Engineer, Foundation Contractor, and Owner to determine if the pile must be repaired or replaced.

Surveillance/Protection

We recommend that photographic and written records be kept of both the pre-existing conditions around the site and new damage (if any) sustained by improvements at the site. The elevation of sidewalks and pavements adjacent to the site should be measured prior to construction activities. The elevations of selected survey points should be measured on a weekly basis during the initial stages of construction. Elevation of improvements and photographs should include basic data for determining the validity of claims lodged by nearby property owners/tenants.

Existing nearby buildings should be monitored for vibration during construction activities to evaluate any adverse responses to vibrations. Should adverse reactions be observed or noted, protection must be provided to the buildings and additional monitoring should be conducted during construction activities. The type of protection should be selected by the contractor based upon the reactions observed.

Interior Floor Slab Support

The following recommendations are applicable if the apartment and/or clubhouse structures are supported on conventional shallow foundations (not PT slab foundations). Interior concrete slab-on-grade floors for the apartment and clubhouse structures can be supported upon very low-expansive, imported soil or lime-treated soil subgrades prepared in accordance with the recommendations included in the Engineered Fill Construction section of this report, provided the subgrade soils are maintained in a moist condition and protected from disturbance.

Concrete slabs-on-grade designated for vehicle support such as those for the planned parking structure should be designed and constructed in accordance with the recommendations provided in the Pavement Design section of this report (e.g. pavement subgrade construction and aggregate base/concrete slab thicknesses).

Interior concrete slab-on-grade floors for the apartment and clubhouse structures should be at least four inches thick. We recommend that interior floor slabs be adequately reinforced to provide structural continuity, mitigate cracking, and permit spanning of local soil irregularities. The project structural engineer should determine final floor slab thickness and reinforcing requirements. Temporary loads exerted during construction from vehicle traffic, construction equipment, storage of palletized construction materials, etc. should be considered in the design of the thickness and reinforcement of the interior slab-on-grade floor.

Floor slabs that will receive moisture sensitive floor covering (e.g. vinyl covering, wood-laminate, etc.) should be underlain by a layer of free-draining gravel/crushed rock, serving as a deterrent to migration of capillary moisture. The gravel/crushed rock layer should be at least four inches thick, but no more than six inches thick, and graded such that 100 percent passes a one-inch sieve and less than five percent passes a No. 4 sieve. Additional moisture protection may be provided by placing a plastic water vapor retarder membrane (at least 10-mils thick) directly over the gravel/crushed rock. The water vapor retarder should meet or exceed the minimum specifications for plastic water vapor retarders as outlined in ASTM E1745 and be installed in strict conformance with the manufacturer's recommendations.

Floor slab construction practice over the past 40 years or more has included placement of a thin layer of sand or pea gravel over the vapor retarder membrane. The intent of the sand/pea gravel is to aid in the proper curing of the slab concrete. However, recent debate over excessive moisture vapor emissions from floor slabs includes concern for water trapped within the sand/pea gravel. As a consequence, we consider the use of the sand/pea gravel layer as optional. The concrete curing benefits should be weighed against efforts to reduce slab moisture vapor transmission.

The recommendations presented above are intended to reduce significant soils-related cracking of slab-on-grade floors. Also important to the performance and appearance of a Portland cement concrete slab is the quality of the concrete, the workmanship of the concrete contractor, the curing techniques utilized and the spacing of control joints.

Floor Slab Moisture Penetration Resistance

It is likely that floor slab subgrade soils will become saturated at some time during the life of the structures, especially when slabs are constructed during the wet seasons, or when constantly wet ground or poor drainage conditions exist adjacent to the buildings. For this reason, it should be assumed that interior slabs require protection against moisture or moisture vapor penetration. Standard practice includes placing a layer of gravel/crushed rock and a vapor retarder membrane (and possibly a layer of sand or pea gravel) as discussed above. Recommendations contained in this report concerning foundation and floor slab design are presented as minimum requirements only from the geotechnical engineering standpoint.

It is emphasized that the use of gravel/crushed rock and plastic membrane below the slab will not "moisture proof" the slab, nor will it assure that slab moisture transmission levels will be low enough to prevent damage to floor coverings or other building components. It is emphasized that we are not slab moisture proofing or moisture protection experts. The sub-slab gravel/crushed rock and vapor retarder membrane simply offers a first line of defense against soil-related moisture. If increased protection against moisture vapor penetration of the slab is desired, a concrete moisture protection specialist should be consulted. The design team should consider all available measures for slab moisture protection. It is commonly accepted that maintaining the lowest practical water-cement ratio in the slab concrete is one of the most effective ways to reduce future moisture vapor penetration of the completed slabs.

Drilled Pier Foundations for Light Standards

Based on our experience with similar projects, we anticipate pole-mounted lights used near walkways will be supported on drilled piers. Drilled piers for support of pole-mounted lights should be no less than 18 inches in diameter and should extend at least five feet below the final soil subgrade elevation.

Drilled piers extending at least five feet below the final soil subgrade elevation may be sized utilizing a maximum allowable vertical bearing capacity of 4,000 psf or an allowable skin friction of 300 psf for dead plus live loads, which may be applied over the surface of the pier. The upper 12 inches of skin friction should be disregarded unless the pier is completely surrounded by concrete or pavements for a distance of at least three feet from the edge of the foundation pier. These values may be increased by one-third to include the short-term wind or seismic forces. The weight of foundation concrete below grade may be disregarded in sizing computations for the end-bearing condition.

Uplift resistance of pier foundations may be computed using the following resisting forces, where applicable: 1) effective weight of the pier concrete (150 pounds per cubic foot), and 2) the allowable skin friction of 300 psf applied over the shaft area of the pier. The upper 12 inches of skin friction should be disregarded unless the pier is completely surrounded by concrete or pavements for a distance of at least three feet from the edge of the foundation pier Increased uplift resistance can be achieved by increasing the diameter of the pier or increasing the depth.

Lateral resistance of pier foundations may be evaluated by applying a passive earth pressure of equivalent to a fluid pressure of 300 psf per foot of depth. The passive pressure may be applied over 1½-pier diameters times the depth of the pier. The upper 12 inches of the subgrade should be disregarded for the non-constrained condition.

The Structural Engineer should determine if reinforcement of the piers is required and determine the reinforcing requirements. The bottom of the pier excavations should not contain loose or disturbed soils prior to placement of the concrete and reinforcement (if required). Cleaning of the bearing surface should be verified by the Geotechnical Engineer’s representative prior to concrete placement. Concrete and reinforcement (if required) should be placed in the pier excavations as soon as possible, after the excavations are completed. The intent of this recommendation is to minimize the chances of sidewall caving into the excavations. Although we do not anticipate excessive sloughing of the sidewalls during pier construction, we recommend that the pier contractor be prepared to case the pier holes if conditions require.

If the drilled piers are constructed in the "dry" (with dry being less than two inches of water at the base of the excavation), the concrete may be placed by the free-fall method, using a short hopper or back-chute to direct the concrete flow out of the truck into a vertical stream of flowing concrete with a relatively small diameter. The stream is directed to avoid hitting the sides of the excavation or any reinforcing cages. For the free-fall method of concrete placement, we recommend the concrete mix be designed with a slump of five to seven inches.

Excavations extending below an elevation of about +13 feet NGVD 29 could encounter groundwater and require temporary dewatering depending on the time of year. If groundwater is encountered, groundwater likely will not be controlled, such that more than six inches of water accumulates at the bottom of the pier excavation. After it is confirmed that the excess water cannot be removed from the drilled pier excavation by bailing or with pumps, concrete should be placed using a tremie. For concrete placed using the tremie method, a design slump of six to eight inches, and a maximum aggregate size of ¾-inch is recommended. The required slump should be obtained by using plasticizers or water-reducing agents. Addition of water on-site to establish the recommended slump should not be allowed.

When extracting temporary casings or tremie methods from drilled pier excavations (if required), care should be taken to maintain a head of concrete to prevent infiltration of water and soil into the shaft area. The head of concrete should always be greater than the head of water trapped outside the pier or tremie, considering the differences in unit weights of concrete and water.

Retaining Walls

The recommendations provided below are applicable to retaining wall not structurally connected to the planned buildings. If retaining walls are structurally connected to the planned buildings, such retaining walls should be evaluated by the Geotechnical Engineer on a case by case basis to determine if the recommendations provided below remain applicable.

Foundations for retaining walls not structurally connected to the planned buildings should be supported on a continuous foundation at least 18 inches wide, extending at least 18 inches below lowest adjacent soil grade. Continuous footings for retaining walls may be designed based on an allowable bearing capacity of 2,500 pounds psf for dead plus live load conditions. The allowable bearing capacity may be increased by one-third for effects of wind or seismic forces.

Retaining walls that will be allowed to slightly rotate about their base (unrestrained at the top or sides) should be capable of resisting "active" lateral earth pressure equal to an equivalent fluid pressure of 50 psf per foot of wall backfill for horizontal backfill and fully drained conditions. Retaining walls that are fixed at the top, such as those for elevator pits and the swimming pool, should be capable of resisting "at-rest" lateral earth pressure equal to an equivalent fluid pressure of 70 psf per foot of wall backfill, again assuming horizontal backfill and fully drained conditions. The equivalent fluid pressures above assume <u>no hydrostatic pressures</u> or surcharge loads behind the wall.

Groundwater at the site could rise to an elevation of about +13 feet NGVD 29. Any retaining walls, such as those for elevator pits and the swimming pool, that extend below an elevation of +13 feet NGVD 29 should be designed to account for hydrostatic pressure caused by groundwater. Retaining structures that extend below an elevation of +13 feet NGVD 29 should be capable of resisting an “active” lateral pressure equal to an equivalent fluid pressure of 80 psf per foot of wall backfill. For “at rest” conditions, retaining structures that extend below an elevation of +13 feet NGVD 29 should be capable of resisting a lateral pressure equal to an equivalent fluid pressure of 90 psf per foot of wall backfill.

Walls supporting sloping backfill, up to a two horizontal to one vertical (2H:1V) inclination, should be designed adding an additional 20 psf per foot of wall to the pressures presented above.

Based on recent research (Lew, et al. 2010), the seismic increment of earth pressures may be neglected if the maximum peak ground acceleration (PGA_M) at the site is 0.4 g or less. Our analysis indicates the PGA_M at the site will be about 0.60 g; therefore, a seismic increment to the lateral earth pressures presented above should be incorporated into the retaining wall design. Earth pressures due to seismic loading may be evaluated by adding a seismic increment of 20 psf per foot of wall backfill to the lateral earth pressures presented above for horizontal backfill conditions. The resultant seismic force should be applied at 1/3 times the height of the retaining wall, measured from the bottom of the wall.

Lateral resistance may be computed using an allowable “passive” earth pressure of 200 psf per foot of depth <u>below</u> an elevation of +13 feet NGVD 29 and 300 psf per foot of depth <u>above</u> an elevation of +13 feet NGVD 29.

Retaining walls, including swimming pool walls, will experience additional surcharge loading if vehicles are parked, equipment is stored, or foundations are within a one horizontal to one vertical (1H:1V) projection from the bottom of the retaining wall. Surcharge loading under these circumstances should be evaluated by the retaining wall or swimming pool designer on a case-by-case basis and be included in the design of the wall, in addition to the lateral earth pressures described above. The surcharge load distribution, magnitude of the surcharge resultant force to be applied on the wall and the location of where the resultant force should be applied will depend on the specific surcharge load type (e.g. point load, distributed load, etc.) and the distance away from the retaining wall.

Backfill behind retaining walls should be fully drained to prevent the build-up of hydrostatic pressures behind the wall. Retaining walls should be provided with a drainage blanket of Class 2 permeable material, *Caltrans Standard Specification, Section 68-2.02F(3)*, at least one foot wide extending from the base of wall to within one foot of the top of the wall. The top foot above the drainage layer should consist of compacted on-site or imported engineered fill materials, unless covered by a concrete slab or pavement. Weep holes or perforated rigid pipe, as appropriate, should be provided at the base of the wall to collect accumulated water. Drainpipes, if used, should slope to discharge at no less than a one percent fall to suitable drainage facilities. Open-graded ½- to ¾-inch crushed rock may be used in lieu of the Class 2 permeable material, if the rock and drainpipe are completely enveloped in an approved non-woven, geotextile filter fabric. Alternatively, approved geotextile drainage composites may be used in lieu of the drain rock layer. If used, geocomposite drain panels should be installed in accordance with the manufacturer's recommendations.

If efflorescence (discoloration of the wall face) or moisture penetration of the wall is not acceptable, waterproofing measures should be applied to the back face of the wall. A specialist in protection against moisture penetration should be consulted to determine specific waterproofing measures.

Structural backfill materials for retaining walls within a one horizontal to one vertical (1H:1V) projection from the bottom of the walls (other than the drainage layer) should consist of very low-expansive (Expansion Index < 20), compactable granular material that does not contain significant quantities of rubbish, rubble, organics and rock over six inches in size. Clay soil, pea gravel and/or crushed rock should not be used for wall backfill. Structural backfill should be placed in lifts not exceeding 12 inches in loose thickness, moisture conditioned to at least the optimum moisture content, and should be mechanically compacted to between 90 and 95 percent relative compaction (ASTM D1557). Over-compaction of structural backfill for retaining walls should be avoided. Backfilling behind retaining walls should not begin until the wall concrete or grout has reached a minimum strength determined by the Structural Engineer.

Swimming Pool Design

The shell or walls for the swimming pool should be designed to resist the lateral earth pressure parameters presented in the Retaining Wall Design section of this report.

Soils exposed following swimming pool wall excavations should be maintained in a moist condition until the construction of the pool walls. The intent of this recommendation is to reduce the potential of desiccation cracking, disturbance, or caving/sloughing of the soils behind the pool walls. The subgrade soils at the pool bottom also should be maintained in a moist condition until the construction of the pool bottom to reduce the potential of heaving of the pool bottom due to swelling of the subgrade soils.

The upper 18 inches of final subgrades to support the swimming pool deck slabs should consist of approved, imported, compactable, very low-expansive (Expansion Index ≤ 20) granular soils or lime-treated on-site or approved imported clay soils placed and compacted in accordance with the <u>Engineered Fill Construction</u> recommendations included in this report. As an alternative, the upper 18 inches of final subgrade for the pool deck slabs could also consist of Class 2 aggregate base uniformly compacted to at least 95 percent relative compaction. The pool deck slabs should be designed to control shrinkage and thermal cracking. The pool deck slab designer should determine and detail the thickness, strength, reinforcement, and joint spacing of the pool deck slab.

A grading plan was not available at the time this report was prepared and the design elevations for the bottom of the swimming pool are unknown; however, we assume that the swimming pool will vary in depth from about three to 10 feet. Excavations extending below an elevation of about +13 feet NGVD 29 could encounter groundwater (depending on the time of year). Therefore, wall for the swimming pool that extend below such elevation should be designed to account for hydrostatic pressure caused by groundwater. Buoyancy effects should be considered by the swimming pool designer in the event the pool is drained and kept empty for an extended period of time. The installation of a hydrostatic relief value at the bottom of the pool in the event the pool is emptied during a period of high groundwater should be considered by the swimming pool designer.

<u>Exterior Flatwork (Non-Pavement Areas)</u>

The upper 12 inches of final soil subgrade for exterior concrete flatwork areas (18 inches for swimming deck slabs) should consist of approved, imported, compactable, very low-expansive (Expansion Index ≤ 20) granular soils or lime-treated on-site or approved imported clay soils placed and compacted in accordance with the <u>Engineered Fill Construction</u> recommendations included in this report. Exterior flatwork subgrade soils should be maintained in a moist condition and protected from disturbance.

Exterior flatwork should be underlain by at least four inches of Class 2 Aggregate Base compacted to at least 95 percent relative compaction. The aggregate base can be included in the 12 inches of very-low expansive granular soils (not lime-treated soils) or the very-low expansive layer can be completely composed of Class 2 Aggregate Base. If the upper 12 inches of final soils subgrade for exterior flatwork will consist of lime-treated clay soils, the four inches of aggregate base should be placed over the lime-treated soils.

Exterior flatwork concrete should be at least four inches thick. Consideration should be given to thickening the edges of the slabs at least twice the slab thickness where wheel traffic is expected over the slabs. Expansion joints should be provided to allow for minor vertical movement of the flatwork. Exterior flatwork should be constructed independent of other structural elements by the placement of a layer of felt material between the flatwork and the structural element. The slab designer should determine the final thickness, strength and joint spacing of exterior slab-on-grade concrete. The slab designer should also determine if slab reinforcement for crack control is required and determine final slab reinforcing requirements.

Areas adjacent to exterior flatwork should be landscaped to maintain more uniform soil moisture conditions adjacent to and under flatwork. We recommend final landscaping plans not allow fallow ground adjacent to exterior concrete flatwork.

Practices recommended by the Portland Cement Association (PCA) for proper placement, curing, joint depth and spacing, construction, and placement of concrete should be followed during exterior concrete flatwork construction.

Site Drainage

Final site grading should be accomplished to provide positive drainage of surface water away from buildings and prevent ponding of water adjacent to foundations, slabs or pavements. The subgrade adjacent to buildings should be sloped away from foundations at a minimum two percent gradient for at least 10 feet, where possible. We recommend connecting all roof drains to solid drainage pipes which are connected to available drainage features to convey water away from the buildings or discharging the drains onto paved or hard surfaces that slope away from the foundations. Discharging or ponding of surface water should not be allowed adjacent to buildings, exterior flatwork or pavements.

Drought Consideration

The soils at the site are considered significantly expansive. These soils swell when the moisture content increases and shrink when the soil moisture content decreases. It will be essential that the soil moisture content under and near foundations and exterior concrete flatwork remain relatively constant to mitigate the potential for heaving or settling of the foundation and slabs.

The State of California can experience significant periods of severe drought conditions. If this occurs, the ability to use irrigation as a means for maintaining landscape vegetation and soil moisture could be inhibited for unpredictable periods of time. For this reason, landscape and hardscape systems should be carefully planned to prevent the desiccation of soils under and near foundations and slabs. Trees with invasive shallow root systems should be avoided. No trees or large shrubs that could remove soil moisture during dry periods should be planted within five feet of any foundation or concrete slab. Fallow ground adjacent to foundations or concrete slabs must be avoided.

Pavement Design

Based on laboratory test results for surface and near-surface clay soils present at the site and the assumption that similar clay soils could be imported to the site to establish final soils subgrade elevations for pavement areas, we used a Resistance (“R”) value of 5 for untreated pavement subgrades and an R-value of 40 for lime-treated pavement subgrades. Pavement sections presented in Table 4 have been calculated using the above R-values and traffic indices (TIs) assumed to be appropriate for this project. The procedures used for pavement design are in general conformance with Chapters 600 to 670 of the *California Highway Design Manual*, dated July 1, 2020. The project civil engineer should determine the appropriate traffic index for pavements based on anticipated traffic conditions. If needed, we can provide additional pavement sections for different traffic indices.

We emphasize that the performance of pavement is critically dependent upon uniform and adequate compaction of the soil subgrade, as well as all engineered fill and utility trench backfill within the limits of the pavements. Final pavement subgrade preparation (i.e. scarification, moisture conditioning and compaction) should be performed after underground utility construction is completed and just prior to aggregate base placement.

The upper six inches of untreated pavement subgrade soils and upper 12 inches of lime-treated subgrade soils should be compacted to at least 95 percent relative compaction at no less than two percent above the optimum moisture content, maintained in that condition (moist) and protected from disturbance. All aggregate base should be compacted to at least 95 relative compaction.

Pavement subgrades should be stable and unyielding under heavy wheel loads of construction equipment. To help identify unstable subgrades within the pavement limits, a proof-roll should be performed with a fully loaded, water truck (at least 4,000 gallons) on the exposed subgrades prior to placement of aggregate base. The proof-roll should be observed by the Geotechnical Engineer’s representative.

Table 4 – On-site Pavement Design Alternatives

Traffic Index (TI)	Pavement Use	Untreated Subgrades R-value = 5			Lime-Treated Subgrades Soils (a) R-value = 40		
		Type A Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	Portland Cement Concrete (inches)	Type A Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	Portland Cement Concrete (inches)
4.5	Automobile Parking Only	2½*	10	--	2½*	4	4
		--	6	4	--	4	4
6.0	Automobile, Light to Moderate Truck Traffic, and Fire Lanes	3	14	--	3	7	--
		3½*	13	--	3½*	6	--
		--	6	6	--	4	6
7.0	Trash Enclosures, and Retail Delivery and Loading Areas	3	18	--	3	9	--
		4*	16	--	4*	7	--
		--	8	7	--	6	7

Notes: * = Asphalt concrete thickness contains the Caltrans safety factor.
(a) = Lime-treated subgrade should be at least 12 inches thick and possess a minimum R-value of 40 when testing in accordance with California Test 301.

In the summer heat, high axle loads coupled with shear stresses induced by sharply turning tire movements can lead to failure in asphalt concrete pavements. Therefore, Portland cement concrete (PCC) pavements should be used in areas subjected to concentrated heavy wheel loading, such as entry driveways, in front of trash enclosures and within retail delivery and loading areas. PCC pavement sections have been provided above in Table 4.

We suggest the concrete slabs be constructed with thickened edges in accordance with American Concrete Institute design standards, latest edition. Reinforcing for crack control, if desired, should be provided in accordance with ACI guidelines. Reinforcement must be located at mid-slab depth to be effective. Joint spacing and details should conform to the current PCA or ACI guidelines. PCC should achieve a minimum compressive strength of 3,500 pounds per square inch at 28 days.

All pavement materials and construction methods of structural pavement sections should conform to the applicable provisions of the *Caltrans Standard Specifications*, latest edition.

Lime-treatment of Pavement Subgrade Soils

On-site or approved imported clay soils are anticipated to react well with the addition of quicklime (high-calcium or dolomitic) and could enhance the support characteristics of the subgrade and allow for a reduction in the aggregate base section. If lime-treatment of subgrade soils is selected, the lime-treatment of subgrade soils should be performed in general conformance with Section 24 of the *Caltrans Standard Specifications*, latest edition. Please note that predominately sandy soils, if encountered, will likely require blending with clayey soils before amendment with quicklime will be effective.

If lime-treatment of pavement subgrades is selected, we recommend a minimum spread rate of at least 4½ pounds of high calcium or dolomitic quicklime per square foot of treated soil, at a depth sufficient to produce a compacted lime-treated layer 12 inches thick. After the materials have been thoroughly mixed and re-mixed, the soil-lime mixture should be compacted to at least 95 percent relative compaction at a moisture content at least two to four percent over optimum conditions. Compaction should be achieved using a heavy, self-propelled sheep's-foot compactor (Rex or equivalent).

Pavement Drainage

Efficient drainage of all surface water to avoid infiltration and saturation of the supporting aggregate base and subgrade soils is important to pavement performance. Weep holes could be provided at drainage inlets, located at the subgrade-base interface, to allow accumulated water to drain from beneath the pavements.
Consideration should be given to using full-depth curbs between landscaped areas and pavements to serve as a cut-off for water that could migrate into the pavement base materials or subgrade soils.

Geotechnical Engineering Observation and Testing During Earthwork Construction

Site preparation should be accomplished in accordance with the recommendations of this report and the *Guide Earthwork Specifications* provided in Appendix E. Geotechnical testing and observation during construction is considered a continuation of our geotechnical engineering investigation. Wallace-Kuhl & Associates should be retained to provide testing and observation services during site clearing, preparation, earthwork, and foundation construction at the project to verify compliance with this geotechnical report and the project plans and specifications, and to provide consultation as required during construction. These services are beyond the scope of work authorized for this study. We would be pleased to submit a proposal to provide these services upon request.

Section 1803.5.8 “Compacted Fill Material” of the *2019 CBC* requires that the geotechnical engineering report provide a number and frequency of field compaction tests to determine compliance with the recommended minimum compaction. Many factors can affect the number of tests that should be performed during the course of construction, such as soil type, soil moisture, season of the year and contractor operations/performance. Therefore, it is crucial that the actual number and frequency of testing be determined by the Geotechnical Engineer during construction based on their observations, site conditions, and construction conditions encountered.

The Geotechnical Engineer of Record (GEOR) for construction shall be retained by the Owner to provide consulting services during construction and whose representatives shall perform required testing and inspection during construction. In the event that our firm is not retained to provide geotechnical engineering observation and testing services during construction, the Geotechnical Engineer retained to provide these services must indicate in writing that they agree with the recommendations of this report, or prepare supplemental recommendations as necessary. A final report by the GEOR retained to provide construction testing services should be prepared upon completion of the project.

Additional Future Services

We recommend that Wallace-Kuhl & Associates be retained to review the final plans and specifications to determine if the intent of our recommendations has been implemented in those documents. We would be pleased to submit a proposal to provide these services upon request.

LIMITATIONS

Our recommendations are based upon the information provided regarding the proposed project, combined with our analysis of site conditions revealed by the field exploration and laboratory testing programs. We have used engineering judgment based upon the information provided and the data generated from our study. This report has been prepared in substantial compliance with generally accepted geotechnical engineering practices that exist in the area of the project at the time the report was prepared. No warranty, either express or implied, is provided.

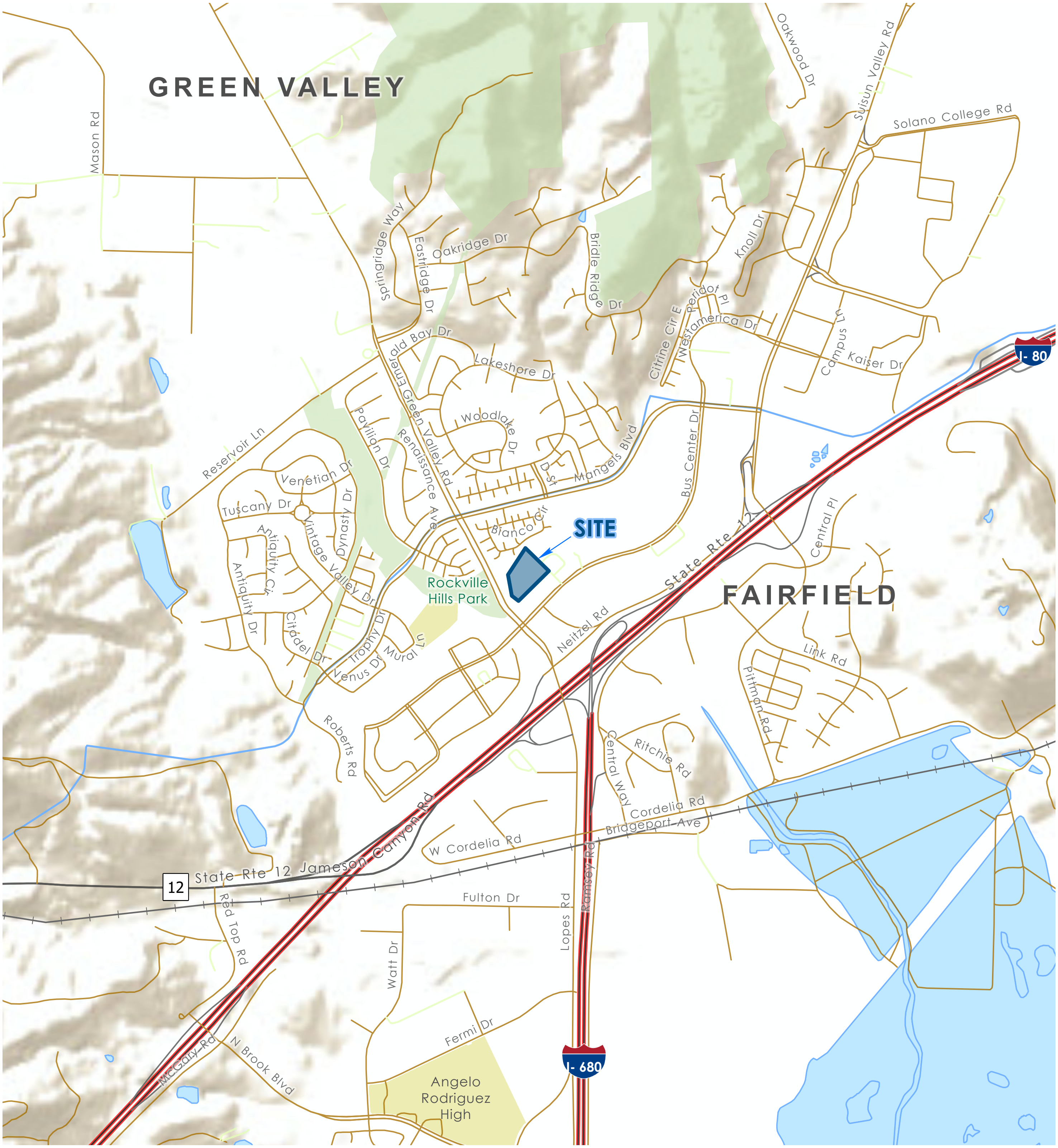
If the proposed construction is modified or re-sited; or, if it is found during construction that subsurface conditions differ from those we encountered at the exploration locations, we should be afforded the opportunity to review the new information or changed conditions to determine if our conclusions and recommendations must be modified.

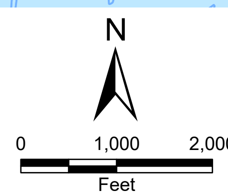
We emphasize that this report is applicable only to the proposed construction and the investigated site and should not be utilized for construction on any other site. The conclusions and recommendations of this report are considered valid for a period of two years. If design is not completed and construction has not started within two years of the date of this report, the report must be reviewed and updated, if necessary.

Wallace - Kuhl & Associates

REGISTERED PROFESSIONAL ENGINEER
DAVID R. GIUS, JR.
NO. 2318
Exp. 9/30/22
GEOTECHNICAL
STATE OF CALIFORNIA

David R. Gius, Jr.
Senior Engineer





Spatial Data provided by Esri, NOAA, and USGS.
Projection: NAD 1983 2011 StatePlane California III FIPS 0403 Ft US

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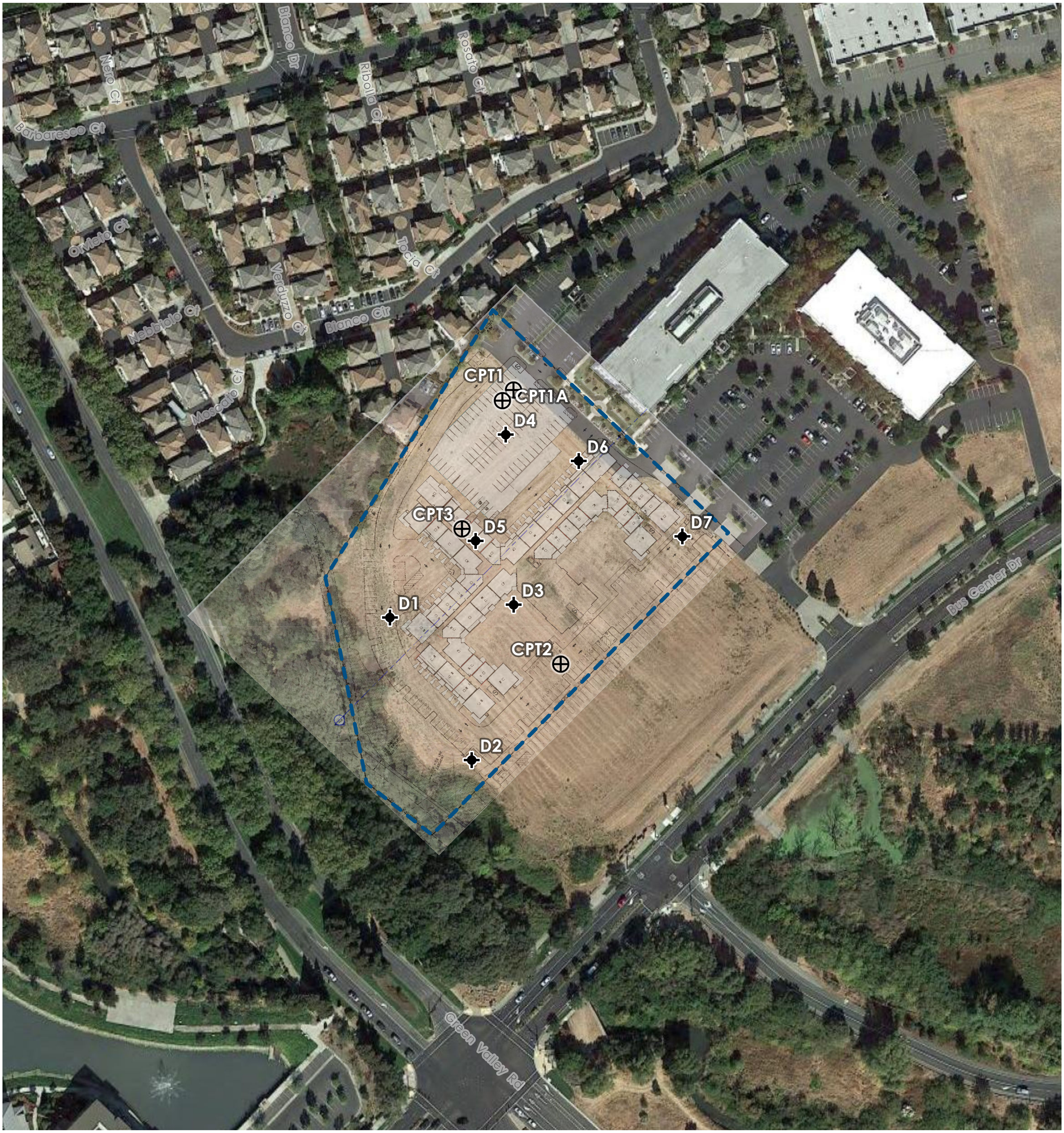
VICINITY MAP

GREEN VALLEY 3 APARTMENTS

Fairfield, California

FIGURE	1
DRAWN BY	RWO
CHECKED BY	TCD
PROJECT MGR	DRG
DATE	02/2022
WKA NO.	13081.02



Approximate Soil Boring Location

Approximate CPT Location

Approximate Site Boundary

Aerial imagery provided by Esri.
Site Plan adapted from a drawing provided by Kepphart, dated 08/12/2021.
Projection: NAD 1983 2011 StatePlane California II FIPS 0402 Ft US

N

0 100 200
Feet

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SITE PLAN

GREEN VALLEY 3 APARTMENTS

Fairfield, California

FIGURE	2
DRAWN BY	RWO
CHECKED BY	TCD
PROJECT MGR	DRG
DATE	02/2022
WKA NO.	13081.02

Project: Green Valley 3 Apartments **Project Location: Fairfield, California** **WKA Number: 13081.02**	**LOG OF SOIL BORING D1** Sheet 1 of 1

Date(s) Drilled	**3/16/21**	Logged By	**GHZ**	Checked By	**ML**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**20.0 feet**
Drill Rig Type	**CME-55 HT**	Diameter(s) of Hole, inches	**6"**	Approx. Surface Elevation, ft MSL	**16.0**
Groundwater Depth [Elevation], feet	**7.0 [9.0]**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks				Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
15			Dark brown, moist, soft, fat CLAY with sand (CH)						
					D1-1I	15	21.2	92	UCC = 0.4 tsf
	5		brown, stiff		D1-2I	16	25.9	94	UCC = 1.0 tsf
10									
	10		dark brown, wet, very stiff, no sand		D1-3I	17	41.1	74	PP = 3.5 tsf
5									
			Gray brown, wet, sandy lean CLAY (CL)						
	15		Brown, wet, medium dense, silty fine to medium SAND (SM)		D1-4I	16	19.9	103	
0									
	20				D1-5I	21			

Boring was terminated at approximately 20 feet below existing ground surface.
Groundwater encountered at approximately 7 feet below existing ground surface.

BORING LOG 13081.02 - GREEN VALLEY 3 APARTMENTS.GPJ WKA.GDT 2/10/22 8:53 AM

Wallace Kuhl & ASSOCIATES

FIGURE 3

Project: Green Valley 3 Apartments **Project Location:** Fairfield, California **WKA Number:** 13081.02	**LOG OF SOIL BORING D2** Sheet 1 of 1

Date(s) Drilled	**3/17/21**	Logged By	**GHZ**	Checked By	**ML**
Drilling Method	**Rotary Wash**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**31.5 feet**
Drill Rig Type	**CME-55 HT**	Diameter(s) of Hole, inches	**4"**	Approx. Surface Elevation, ft MSL	**17.0**
Groundwater Depth [Elevation], feet	**7.0 [10.0]**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks	**Bulk(0'-3')**			Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet: 15, 10, 5, 0, -5, -10

DEPTH, feet: 5, 10, 15, 20, 25, 30

ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
Dark brown, moist, very stiff, sandy fat CLAY (CH)	D2(0-3')				
	D2-1I	23	19.2	84	PP = 4.5+ tsf
	D2-2I	18	30.8	81	PP = 2.5 tsf
gray, wet, stiff	D2-3I	13	27.9	93	PP = 1.75 tsf
dark brown, medium stiff	D2-4I	7	59.7	63	PP = 1.5 tsf
stiff	D2-5I	14	39.5	79	PP = 2.5 tsf
Brown, wet, medium dense, silty fine to coarse SAND with fine gravel (SM)	D2-6I	36			
	D2-7I	27			

Boring was terminated at approximately 31.5 feet below existing ground surface.
Groundwater encountered at approximately 7 feet below existing ground surface.

Project: Green Valley 3 Apartments
Project Location: Fairfield, California
WKA Number: 13081.02

LOG OF SOIL BORING D3

Date(s) Drilled	**3/17/21**	Logged By	**GHZ**	Checked By	**ML**
Drilling Method	**Rotary Wash**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**51.5 feet**
Drill Rig Type	**CME-55 HT**	Diameter(s) of Hole, inches	**4"**	Approx. Surface Elevation, ft MSL	**19.0**
Groundwater Depth [Elevation], feet	**8.0 [11.0]**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks	**Bulk(0'-3')**			Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
	0	Dark brown, moist, very stiff, fat CLAY (CH)	D3(0-3')				
15			D3-1I	22	18.0	93	PP = 4.5+ tsf
	5	stiff	D3-2I	13	32.7	82	PP = 2.5 tsf
10	10	Olive brown with red motling, wet, medium stiff, sandy lean CLAY (CL)	D3-3I	7	31.1	89	UCC = 0.6 tsf
5		Dark brown, wet, stiff, fat CLAY (CH)					
	15		D3-4I	12	53.3	67	PP = 2.5 tsf
0	20	medium stiff	D3-5I	5	57.2	66	UCC = 0.5 tsf
-5	25		D3-6I	7	37.5	81	PP = 1.75 tsf
		Dark brown, wet, medium dense, silty fine to coarse SAND with fine gravel (SM)					
-10	30	Brown, wet, very stiff, lean CLAY (CL); variably cemented	D3-7I	35	36.6	81	PP = 4.5+ tsf
-15	35	with fine sand	D3-8I	26	38.9	75	PP = 4.5+ tsf
-20		Brown, wet, dense, silty fine to coarse SAND with fine gravel (SM)					
	40		D3-9I	36	28.3	86	
		Brown, wet, lean CLAY (CL)					
-25		Brown, wet, dense, silty fine SAND (SM)					
	45		D3-10I	39			
-30		Brown, wet, medium dense, clayey fine to coarse SAND with fine gravel (SC)					
	50		D3-11I	33			

Boring was terminated at approximately 51½ feet below existing ground surface.
Groundwater encountered at approximately 8 feet below existing ground surface.

BORING LOG 13081.02 - GREEN VALLEY 3 APARTMENTS.GPJ WKA.GDT 2/10/22 8:53 AM

Wallace Kuhl & ASSOCIATES

FIGURE 5

Project: Green Valley 3 Apartments
Project Location: Fairfield, California
WKA Number: 13081.02

LOG OF SOIL BORING D4

Sheet 1 of 1

Date(s) Drilled	3/16/21	Logged By	GHZ	Checked By	ML
Drilling Method	Solid Flight Auger	Drilling Contractor	V&W Drilling	Total Depth of Drill Hole	16.5 feet
Drill Rig Type	CME-55 HT	Diameter(s) of Hole, inches	6"	Approx. Surface Elevation, ft MSL	18.0
Groundwater Depth [Elevation], feet	8.0 [10.0]	Sampling Method(s)	2.0" Modified California with 6-inch sleeve	Drill Hole Backfill	Neat Cement
Remarks	Bulk(0'-3'); 96%<#200; PI = 34; EI = 123			Driving Method and Drop	140lb auto. hammer with 30" drop

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Dark brown, moist, very stiff, fat CLAY (CH)		D4(0-3')				
15					D4-1I	17	26.4	84	PP = 3.5tsf
	5		brown with red motling, stiff		D4-2I	14	32.9	80	PP = 2.0 tsf
10									
	10				D4-3I	16	50.3	69	PP = 2.75 tsf
5									
	15				D4-4I	14	45.5	73	PP = 2.5 tsf

Boring was terminated at approximately 16½ feet below existing ground surface.
Groundwater encountered at approximately 8 feet below existing ground surface.

BORING LOG 13081.02 - GREEN VALLEY 3 APARTMENTS.GPJ WKA.GDT 2/10/22 8:53 AM

Wallace Kuhl & ASSOCIATES

Project: Green Valley 3 Apartments
Project Location: Fairfield, California
WKA Number: 13081.02

LOG OF SOIL BORING D5

Date(s) Drilled	**3/17/21**	Logged By	**GHZ**	Checked By	**ML**
Drilling Method	**Rotary Wash**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**30.0 feet**
Drill Rig Type	**CME-55 HT**	Diameter(s) of Hole, inches	**4"**	Approx. Surface Elevation, ft MSL	**19.0**
Groundwater Depth [Elevation], feet	**8.0 [11.0]**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks	**Bulk(0'-3')**			Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Dark brown, moist, very stiff, sandy fat CLAY (CH)		D5(0-3')				
					D5-1I	29	18.0	95	PP = 4.5+ tsf
15	5		with sand		D5-2I	26	32.6	79	PP = 3.25 tsf
10	10		brown with gray mottling, wet, stiff		D5-3I	8	37.9	80	UCC = 1.1 tsf
5	15		dark brown, very stiff		D5-4I	17	43.5	70	PP = 2.5 tsf
0	20		dark gray, soft, increased sand content		D5-5I	6	43.0	74	UCC = 0.4 tsf
			Brown, wet, loose, silty fine SAND (SM); trace of clay						
-5	25				D5-6I	11	35.9	83	
-10			with fine gravel						
	30		Brown, wet, hard, sandy lean CLAY (CL)		D5-7I	44	42.1	77	

Boring was terminated at approximately 30 feet below existing ground surface.
Groundwater encountered at approximately 8 feet below existing ground surface.

Project: Green Valley 3 Apartments
Project Location: Fairfield, California
WKA Number: 13081.02

LOG OF SOIL BORING D6

Date(s) Drilled	**3/16/21**	Logged By	**GHZ**	Checked By	**ML**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**20.0 feet**
Drill Rig Type	**CME-55 HT**	Diameter(s) of Hole, inches	**6"**	Approx. Surface Elevation, ft MSL	**19.0**
Groundwater Depth [Elevation], feet	**7.0 [12.0]**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks				Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			Dark brown, moist, stiff, fat CLAY (CH)						
					D6-1I	14	20.9	89	UCC = 1.6 tsf
15	5				D6-2I	15	31.2	80	UCC = 1.2 tsf
			Dark brown, wet, clayey fine to coarse SAND with fine gravel (SC)						
10	10		Gray brown, wet, stiff, lean CLAY (CL)		D6-3I	11	38.5	79	PP = 3.25 tsf
			Gray brown, wet, silty fine to coarse SAND with fine gravel (SC)						
5	15		Gray brown, wet, very stiff, sandy lean CLAY (CL)		D6-4I	19	34.3	84	
			Brown, wet, medium dense, silty fine to coarse SAND with fine gravel (SM)						
0	20				D6-5I	28			

Boring was terminated at approximately 20 feet below existing ground surface.
Groundwater encountered at approximately 7 feet below existing ground surface.

Project: Green Valley 3 Apartments
Project Location: Fairfield, California
WKA Number: 13081.02

LOG OF SOIL BORING D7

Date(s) Drilled	**3/16/21**	Logged By	**GHZ**	Checked By	**ML**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**21.5 feet**
Drill Rig Type	**CME-55 HT**	Diameter(s) of Hole, inches	**6"**	Approx. Surface Elevation, ft MSL	**17.0**
Groundwater Depth [Elevation], feet	**7.0 [10.0]**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks	**Bulk(0'-3'); 86%<#200; PI = 40; EI = 121**			Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
17	0	Dark brown, moist, stiff, fat CLAY with sand (CH)	D7(0-3')				
15			D7-1I	12	35.6	78	UCC = 1.2 tsf
	5	olive brown with red motling	D7-2I	12	33.3	85	UCC = 1.2 tsf
10							
	10	dark brown, wet	D7-3I	16	39.4	79	PP = 3.25 tsf
5							
	15		D7-4I	16	33.7	77	PP = 4.5+ tsf
0							
	20	Dark brown, wet, medium dense, clayey fine to medium SAND with fine gravel (SC)	D7-5I	22			

Boring was terminated at approximately 21.5 feet below existing ground surface.
Groundwater encountered at approximately 7 feet below existing ground surface.

Wallace Kuhl & ASSOCIATES

FIGURE 9

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D2487)

MAJOR DIVISIONS		USCS[4]	CODE	CHARACTERISTICS
COARSE GRAINED SOILS (More than 50% of soil > no. 200 sieve size)	GRAVELS[1] (More than 50% of coarse fraction > no. 4 sieve size)	GW		Well-graded gravels or gravel - sand mixtures, trace or no fines
		GP		Poorly graded gravels or gravel - sand mixtures, trace or no fines
		GM		Silty gravels, gravel - sand - silt mixtures, containing little to some fines[2]
		GC		Clayey gravels, gravel - sand - clay mixtures, containing little to some fines[2]
	SANDS[1] (50% or more of coarse fraction < no. 4 sieve size)	SW		Well-graded sands or sand - gravel mixtures, trace or no fines
		SP		Poorly graded sands or sand - gravel mixtures, trace or no fines
		SM		Silty sands, sand - gravel - silt mixtures, containing little to some fines[2]
		SC		Clayey sands, sand - gravel - clay mixtures, containing little to some fines[2]
FINE GRAINED SOILS (50% or more of soil < no. 200 sieve size)	SILTS & CLAYS LL < 50	ML		Inorganic silts, gravely silts, and sandy silts that are non-plastic or with low plasticity
		CL		Inorganic lean clays, gravelly lean clays, sandy lean clays of low to medium plasticity[3]
		OL		Organic silts, organic lean clays, and organic silty clays
	SILTS & CLAYS LL ≥ 50	MH		Inorganic elastic silts, gravelly elastic silts, and sandy elastic silts
		CH		Inorganic fat clays, gravelly fat clays, sandy fat clays of medium to high plasticity
		OH		Organic fat clays, gravelly fat clays, sandy fat clays of medium to high plasticity
HIGHLY ORGANIC SOILS		PT		Peat
ROCK		RX		Rocks, weathered to fresh
FILL		FILL		Artificially placed fill material

OTHER SYMBOLS

= Drive Sample: 2-1/2" O.D. Modified California sampler

= Drive Sampler: no recovery

= SPT Sampler

= Initial Water Level

= Final Water Level

— — — = Estimated or gradational material change line

———— = Observed material change line

Laboratory Tests

- CR = Corrosion
- PI = Plasticity Index
- EI = Expansion Index
- UCC = Unconfined Compression Test (TSF)
- TR = Triaxial Compression Test
- GR = Gradational Analysis (Sieve/Hydro)
- FC = Wash (Fines Content)
- PP = Pocket Penetrometer Test (TSF)
- PID = Photo Ionization Detector Test (PPM)
- RV = Resistance ("R") Value

REF = Refusal (>50 blows in 6 inches)

GRAIN SIZE CLASSIFICATION

CLASSIFICATION	RANGE OF GRAIN SIZES	
	U.S. Standard Sieve Size	Grain Size in Millimeters
BOULDERS (b)	Above 12"	Above 300
COBBLES (c)	12" to 3"	300 to 75
GRAVEL (g)	3" to No. 4	75 to 4.75
coarse	3" to 3/4"	75 to 19
fine	3/4" to No. 4	19 to 4.75
SAND	No. 4 to No. 200	4.75 to 0.075
coarse	No. 4 to No. 10	4.75 to 2.00
medium	No. 10 to No. 40	2.00 to 0.425
fine	No. 40 to No. 200	0.425 to 0.075
SILT & CLAY	Below No. 200	Below 0.075

Trace - Less than 5 percent
Few - 5 to 10 percent
Little - 15 to 25 percent
Some - 35 to 45 percent
Mostly - 50 to 100 percent

* Percents as given in ASTM D2488

NOTES:

1. Coarse grained soils containing 5% to 12% fines, use dual classification symbol (ex. SP-SM).
2. If fines classify as CL-ML (4<PI<7), use dual symbol (ex. SC-SM).
3. Silty Clays, use dual symbol (CL-ML).
4. Borderline soils with uncertain classification list both classifications (ex. CL/ML).

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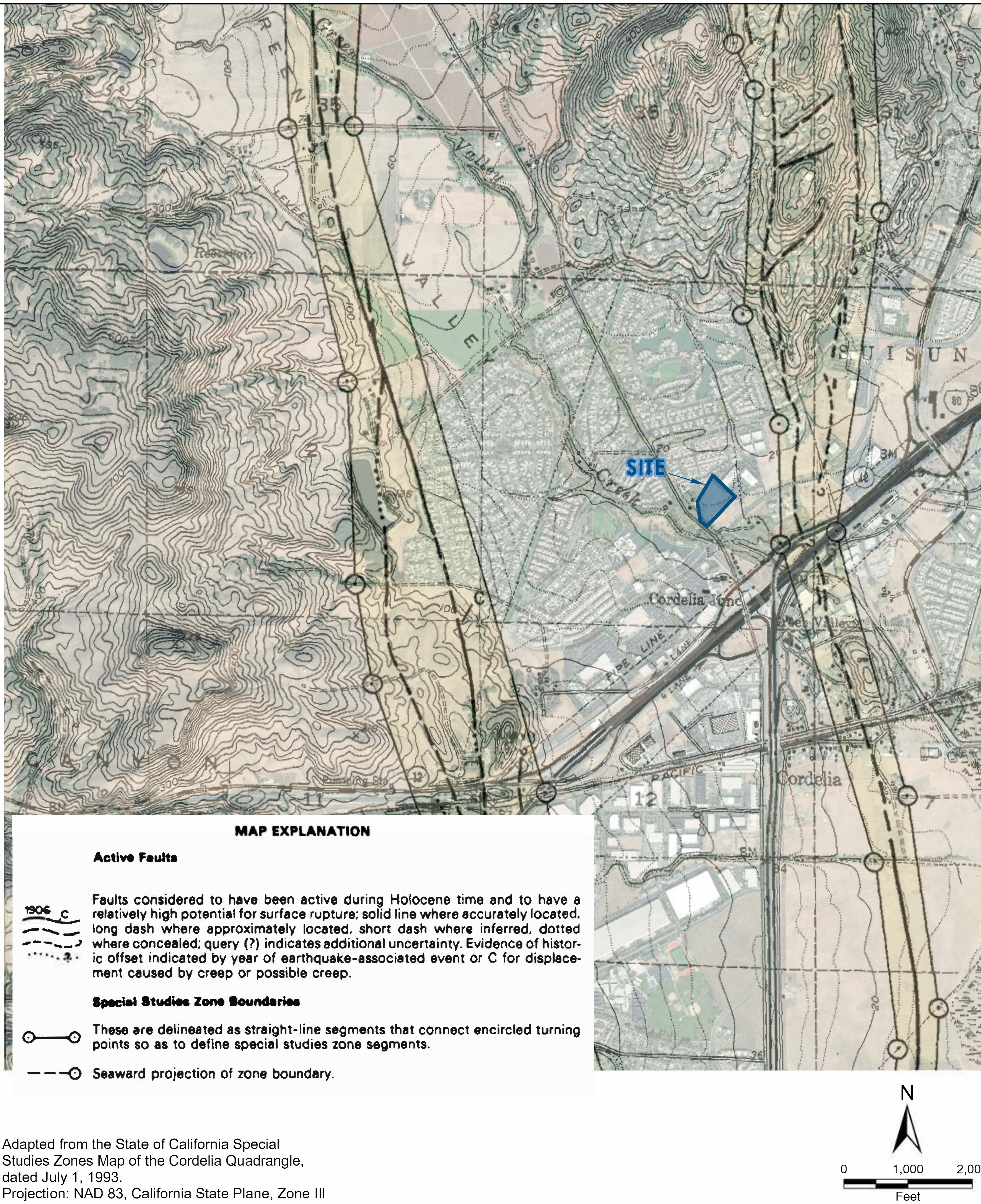
UNIFIED SOIL CLASSIFICATION SYSTEM

GREEN VALLEY 3 APARTMENTS

Fairfield, California

FIGURE	10
DRAWN BY	RWO
CHECKED BY	TCD
PROJECT MGR	DRG
DATE	02/2022
WKA NO.	13081.02



Adapted from the State of California Special Studies Zones Map of the Cordelia Quadrangle, dated July 1, 1993.
Projection: NAD 83, California State Plane, Zone III

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SPECIAL STUDY ZONES MAP

GREEN VALLEY 3 APARTMENTS

Fairfield, California

FIGURE	11
DRAWN BY	RWO
CHECKED BY	TCD
PROJECT MGR	DRG
DATE	02/2022
WKA NO.	13081.02

APPENDICES

APPENDIX A
General Project Information, Laboratory Testing and Results

APPENDIX A

A. GENERAL INFORMATION

The performance of a geotechnical engineering study for the proposed apartment complex to be constructed at 4840 Business Center Drive in Fairfield, California in Sacramento, California, was verbally authorized by Mr. Nicolas Ruhl of The Spanos Corporation on February 25, 2021. Authorization was for an investigation as described in our proposal letter dated February 24, sent to our client The Spanos Corporation whose address is 10100 Trinity Parkway, 5th floor in Stockton, California 95219, telephone (209) 478-7954.

In performing this study, we referred to the “Floor Plan Level 02” prepared by Kephart Architects, dated August 12, 2021.

B. FIELD EXPLORATION

As part of our study, the field exploration program included the drilling and sampling of Seven borings (D1 through D7) and the advancement of four cone penetrometer test (CPT) soundings (CPT1, CPT1A, CPT2 and CPT3) at the approximate locations shown on Figure 2. The boring locations were determined using a previous version of the site development plan.

The seven borings were drilled at the site on March 16 and 17, 2021, by utilizing a CME-55 High Torque truck-mounted, drill rig equipped with six-inch-diameter, solid stem augers and mud-rotary drilling equipment. The borings were drilled to depths ranging from approximately 16½ to 51½ feet below existing site grades. At various intervals soil samples were recovered with a 2½-inch-outside diameter, 2-inch-inside diameter, modified California split-spoon sampler. The sampler was driven by an automatic 140-pound hammer freely falling 30 inches. The number of blows of the hammer required to drive the 18-inch long sampler each 6-inch interval was recorded. The sum of the blows required to drive the sampler the lower 12-inch interval is designated the penetration resistance or "blow count" for that particular drive. The modified California samples were retained in 2-inch-diameter by 6-inch-long, thin walled brass tubes contained within the sampler. After recovery, the field representative visually classified the soil recovered in the tubes. After the samples were classified, the ends of the tubes were sealed to preserve the natural moisture contents.

In addition to the driven samples from the borings, representative bulk samples of near-surface soils also were collected and retained in plastic bags. Driven and bulk samples were taken to our laboratory for additional soil classification and selection of samples for testing.

The four CPT soundings were advanced at the site on March 17, 2021 by utilizing a 25-ton, truck-mounted CPT rig provided by Middle Earth Geo Testing, Inc. of Orange, California. The CPT consisted of advancing a 15-square-centimeter cone penetrometer at a rate of about one inch per second to depths ranging from about 41 to 101 feet below existing site grades. Data was collected from the CPT soundings at an approximate depth interval of two inches. Shear wave velocity data was collected from CPT2 at an approximate depth interval of 5 feet.

The Logs of Soil Borings, Figures 3 through 09, contain descriptions of the soils encountered at each boring location. A boring legend explaining the Unified Soil Classification System and the symbols used on the logs is contained on Figure 10. Copies of the reports for CPT1, provided by Middle Earth Geo Testing, Inc. are included in Appendix B.

C. LABORATORY TESTING

Selected undisturbed samples of the soils were tested to determine dry unit weight (ASTM D2937), natural moisture content (ASTM D2216) and unconfined compression strengths (ASTM D2166). The results of these tests are included in the Logs of Borings at the depth each sample was obtained.

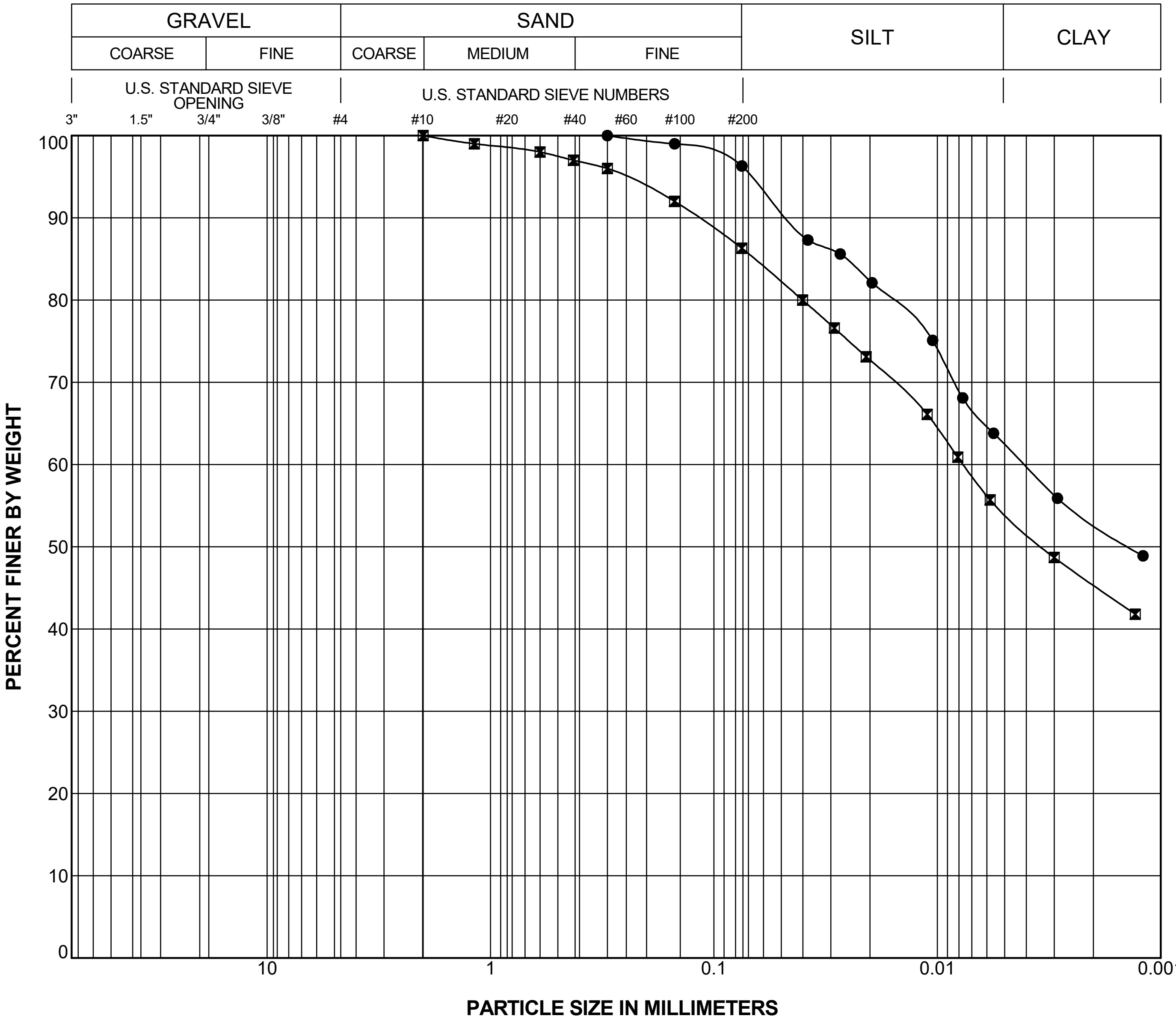
Two samples of surface and near-surface soils was tested for particle-size distribution (ASTM D6913 and ASTM D7928). The results of the particle-size distribution tests are contained in Figure A1.

Two samples of surface and near-surface soils was subjected to Atterberg limits tests (ASTM D4318). The results of these tests are presented in Figure A2.

Two samples of surface and near-surface soils were subjected to Expansion Index testing (ASTM D4829). The results of these tests are presented in Figures A3 and A4.

One bulk sample of anticipated pavement subgrade soils (untreated) and one bulk sample of anticipated pavement subgrade soils mixed with about four-percent dolomitic quicklime were subjected to Resistance ("R") value testing in accordance with California Test 301. The results of the R-value tests, which were used in the pavement design, are presented in Figure A5.

Three samples of surface and near-surface soils were submitted to Sunland Analytical Lab, Inc. of Rancho Cordova, California to determine the soil pH and minimum resistivity (California Test 643), Sulfate concentration (California Test 417 and ASTM D516) and Chloride concentration (California Test 422). The results of these tests are presented in Figures A6 through A11.



Boring Number	Sample Number	USCS	Depth (feet)	Symbol	LL	PI	Classification
D4	D4 (0-3')	CH	0-3'	●	63	34	Dark brown, fat CLAY
D7	D7 (0-3')	CH	0-3'	⊠	64	40	Dark brown, fat CLAY with sand

PARTICLE SIZE DISTRIBUTION

Project: Business Center Drive Apartments
WKA No. 13081.02

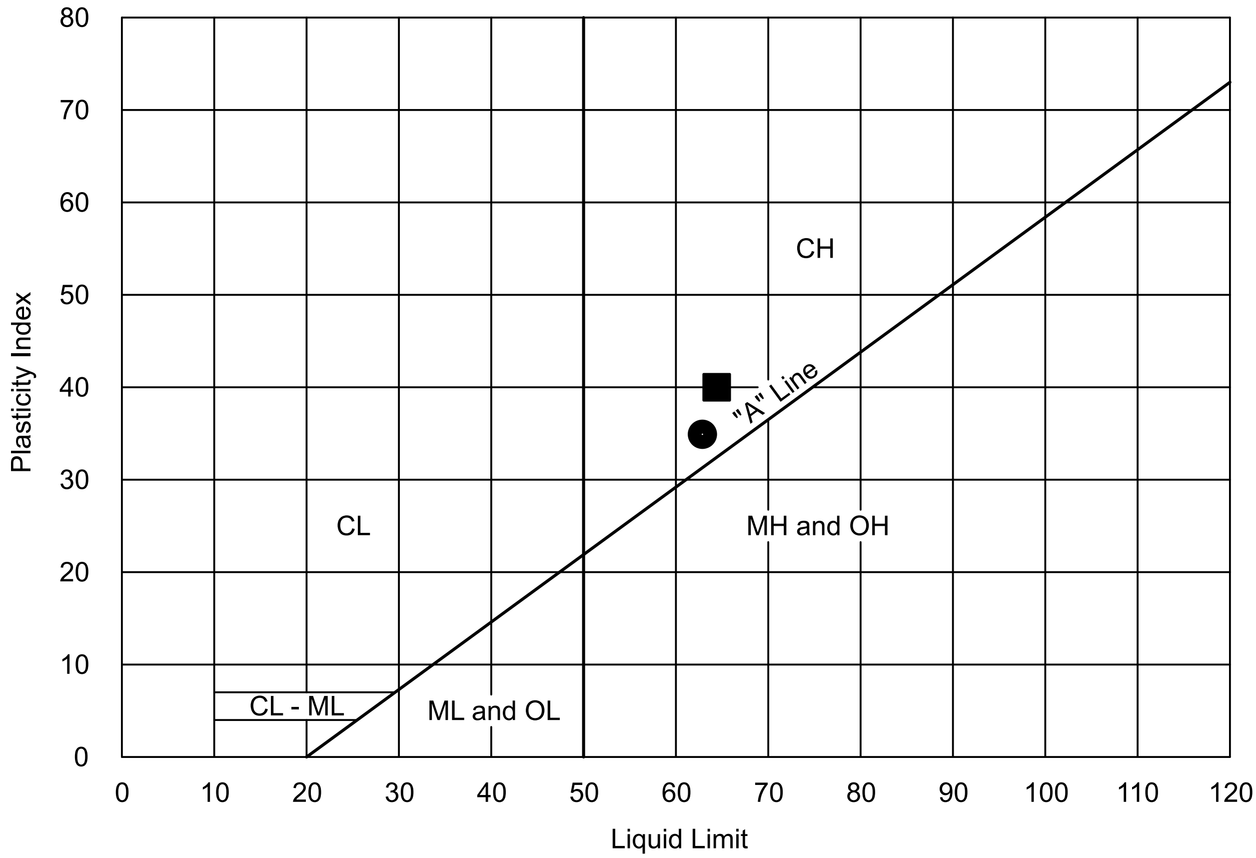
FIGURE A1

GRAIN SIZE 13081.02 - BUSINESS CENTER DRIVE APARTMENTS.GPJ WKA.GDT 4/2/21 1:33 PM

Wallace Kuhl & ASSOCIATES

ATTERBERG LIMITS

ASTM D4318



KEY SYMBOL	LOCATION	SAMPLE DEPTH	NATURAL WATER CONTENT (%)	ATTERBERG LIMITS		PASSING No. 200 SIEVE (%)	UNIFIED SOIL CLASSIFI-CATION SYMBOL
				LIQUID LIMIT (%)	PLASTICITY INDEX (%)		
●	D4	0'-3'	---	63	34	96	CH
■	D7	0'-3'	---	64	40	86	CH

A Universal Engineering Sciences Company

WallaceKuhl & ASSOCIATES

ATTERBERG LIMITS

GREEN VALLEY 3 APARTMENTS

Fairfield, California

FIGURE	A2
DRAWN BY	RWO
CHECKED BY	TCD
PROJECT MGR	DRG
DATE	02/2022
WKA NO. 13081.02	

EXPANSION INDEX TEST RESULTS

ASTM D4829

MATERIAL DESCRIPTION: Dark brown, fat clay

LOCATION: D4

Sample Depth	Pre-Test Moisture (%)	Post-Test Moisture (%)	Dry Density (pcf)	Expansion Index
0' - 3'	19.3	44.0	83	**123**

CLASSIFICATION OF EXPANSIVE SOIL *

EXPANSION INDEX	POTENTIAL EXPANSION
0 - 20	Very Low
21 - 50	Low
51 - 90	Medium
91 - 130	**High**
Above 130	Very High

* From ASTM D4829, Table 1

WallaceKuhl & ASSOCIATES — A Universal Engineering Sciences Company

EXPANSION INDEX

GREEN VALLEY 3 APARTMENTS

Fairfield, California

FIGURE	A3
DRAWN BY	RWO
CHECKED BY	TCD
PROJECT MGR	DRG
DATE	02/2022

WKA NO. 13081.02

EXPANSION INDEX TEST RESULTS

ASTM D4829

MATERIAL DESCRIPTION: Dark brown, fat clay with sand

LOCATION: D7

Sample Depth	Pre-Test Moisture (%)	Post-Test Moisture (%)	Dry Density (pcf)	Expansion Index
0' - 3'	16.5	37.1	89	**121**

CLASSIFICATION OF EXPANSIVE SOIL *

EXPANSION INDEX	POTENTIAL EXPANSION
0 - 20	Very Low
21 - 50	Low
51 - 90	Medium
91 - 130	**High**
Above 130	Very High

* From ASTM D4829, Table 1

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A Universal Engineering Sciences Company

EXPANSION INDEX

GREEN VALLEY 3 APARTMENTS

Fairfield, California

FIGURE	A4
DRAWN BY	RWO
CHECKED BY	TCD
PROJECT MGR	DRG
DATE	02/2022
WKA NO. 13081.02	

RESISTANCE VALUE TEST RESULTS

(California Test 301)

MATERIAL DESCRIPTION: Dark brown, sandy fat clay

LOCATION: D5 (0' - 3')

Specimen No.	Dry Unit Weight (pcf)	Moisture @ Compaction (%)	Exudation Pressure (psi)	Expansion (dial, inches x 1000)	Expansion (psf)	R Value
1	103	20.6	654	40	173	*

*Sample extruded, therefore R-Value = 5

MATERIAL DESCRIPTION: Dark brown, fat clay with about 4% lime

LOCATION: D3 (0' - 3')

Specimen No.	Dry Unit Weight (pcf)	Moisture @ Compaction (%)	Exudation Pressure (psi)	Expansion (dial, inches x 1000)	Expansion (psf)	R Value
1	101	20.1	489	0	0	91
2	101	21.9	384	0	0	89
3	100	22.7	259	0	0	86

R-Value at 300 psi exudation pressure = 87

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RESISTANCE VALUE TEST RESULTS

GREEN VALLEY 3 APARTMENTS

Fairfield, California

FIGURE	A5
DRAWN BY	RWO
CHECKED BY	TCD
PROJECT MGR	DRG
DATE	02/2022
WKA NO.	13081.02

Sunland Analytical

11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 03/31/2021
Date Submitted 03/26/2021

To: Mauricio Luna
Wallace-Kuhl & Assoc.
3050 Industrial Blvd.
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D. \ Randy Horney RH
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : BUISNESS CENTER DR. Site ID : D4 (0-3).
Thank you for your business.

* For future reference to this analysis please use SUN # 84386-175946.

EVALUATION FOR SOIL CORROSION

Soil pH	6.84	
Minimum Resistivity	0.64 ohm-cm (x1000)	
Chloride	100.3 ppm	00.01003 %
Sulfate	181.1 ppm	00.01811 %

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422m

WallaceKuhl & ASSOCIATES | A Universal Engineering Sciences Company

CORROSION TEST RESULTS

GREEN VALLEY 3 APARTMENTS

Fairfield, California

FIGURE	A6
DRAWN BY	RWO
CHECKED BY	TCD
PROJECT MGR	DRG
DATE	02/2022
WKA NO.	13081.02

Sunland Analytical

11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 03/31/2021
Date Submitted 03/26/2021

To: Mauricio Luna
Wallace-Kuhl & Assoc.
3050 Industrial Blvd.
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D. \ Randy Horney RH
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : BUISNESS CENTER DR. Site ID : D4 (0-3).
Thank you for your business.

* For future reference to this analysis please use SUN # 84386-175947.

Extractable Sulfate in Water

Type of TEST	Result	Units
Sulfate-SO4	190.5	mg/kg

METHODS
ASTM D-516m from sat.paste extract-reported based on dry wt.

WallaceKuhl & Associates — A Universal Engineering Sciences Company	CORROSION TEST RESULTS GREEN VALLEY 3 APARTMENTS Fairfield, California	FIGURE A7
		DRAWN BY: RWO
		CHECKED BY: TCD
		PROJECT MGR: DRG
		DATE: 02/2022
		WKA NO. 13081.02

Sunland Analytical

11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 03/31/2021
Date Submitted 03/26/2021

To: Mauricio Luna
Wallace-Kuhl & Assoc.
3050 Industrial Blvd.
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : BUISNESS CENTER DR. Site ID : D7 (0-3).
Thank you for your business.

* For future reference to this analysis please use SUN # 84386-175948.

EVALUATION FOR SOIL CORROSION

Soil pH	6.89		
Minimum Resistivity	0.38	ohm-cm (x1000)	
Chloride	77.4 ppm	00.00774	%
Sulfate	411.0 ppm	00.04110	%

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422m

WallaceKuhl & ASSOCIATES — A Universal Engineering Sciences Company	**CORROSION TEST RESULTS** GREEN VALLEY 3 APARTMENTS Fairfield, California	FIGURE	A8
		DRAWN BY	RWO
		CHECKED BY	TCD
		PROJECT MGR	DRG
		DATE	02/2022
		WKA NO.	13081.02

Sunland Analytical

11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 03/31/2021
Date Submitted 03/26/2021

To: Mauricio Luna
Wallace-Kuhl & Assoc.
3050 Industrial Blvd.
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : BUISNESS CENTER DR. Site ID : D7 (0-3).
Thank you for your business.

* For future reference to this analysis please use SUN # 84386-175949.

Extractable Sulfate in Water

Type of TEST	Result	Units
Sulfate-SO4	427.0	mg/kg

METHODS
ASTM D-516m from sat.paste extract-reported based on dry wt.

WallaceKuhl & Associates — A Universal Engineering Sciences Company

CORROSION TEST RESULTS

GREEN VALLEY 3 APARTMENTS

Fairfield, California

FIGURE	A9
DRAWN BY	RWO
CHECKED BY	TCD
PROJECT MGR	DRG
DATE	02/2022
WKA NO.	13081.02

Sunland Analytical

11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 03/31/2021
Date Submitted 03/26/2021

To: Mauricio Luna
Wallace-Kuhl & Assoc.
3050 Industrial Blvd.
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : BUISNESS CENTER DR. Site ID : D3-4II.
Thank you for your business.

* For future reference to this analysis please use SUN # 84386-175950.

EVALUATION FOR SOIL CORROSION

Soil pH	7.19	
Minimum Resistivity	0.67 ohm-cm (x1000)	
Chloride	7.8 ppm	00.00078 %
Sulfate	33.6 ppm	00.00336 %

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422m

WallaceKuhl & ASSOCIATES
A Universal Engineering Sciences Company

CORROSION TEST RESULTS

GREEN VALLEY 3 APARTMENTS

Fairfield, California

FIGURE	A10
DRAWN BY	RWO
CHECKED BY	TCD
PROJECT MGR	DRG
DATE	02/2022
WKA NO.	13081.02

Sunland Analytical

11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 03/31/2021
Date Submitted 03/26/2021

To: Mauricio Luna
Wallace-Kuhl & Assoc.
3050 Industrial Blvd.
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : BUISNESS CENTER DR. Site ID : D3-4II .
Thank you for your business.

* For future reference to this analysis please use SUN # 84386-175951.

Extractable Sulfate in Water

Type of TEST	Result	Units
Sulfate-SO4	36.5	mg/kg

METHODS
ASTM D-516m from sat.paste extract-reported based on dry wt.

WallaceKuhl & ASSOCIATES
A Universal Engineering Sciences Company

CORROSION TEST RESULTS

GREEN VALLEY 3 APARTMENTS

Fairfield, California

FIGURE	A11
DRAWN BY	RWO
CHECKED BY	TCD
PROJECT MGR	DRG
DATE	02/2022
WKA NO.	13081.02

APPENDIX B
CPT Reports by Middle Earth Geo Testing, Inc.

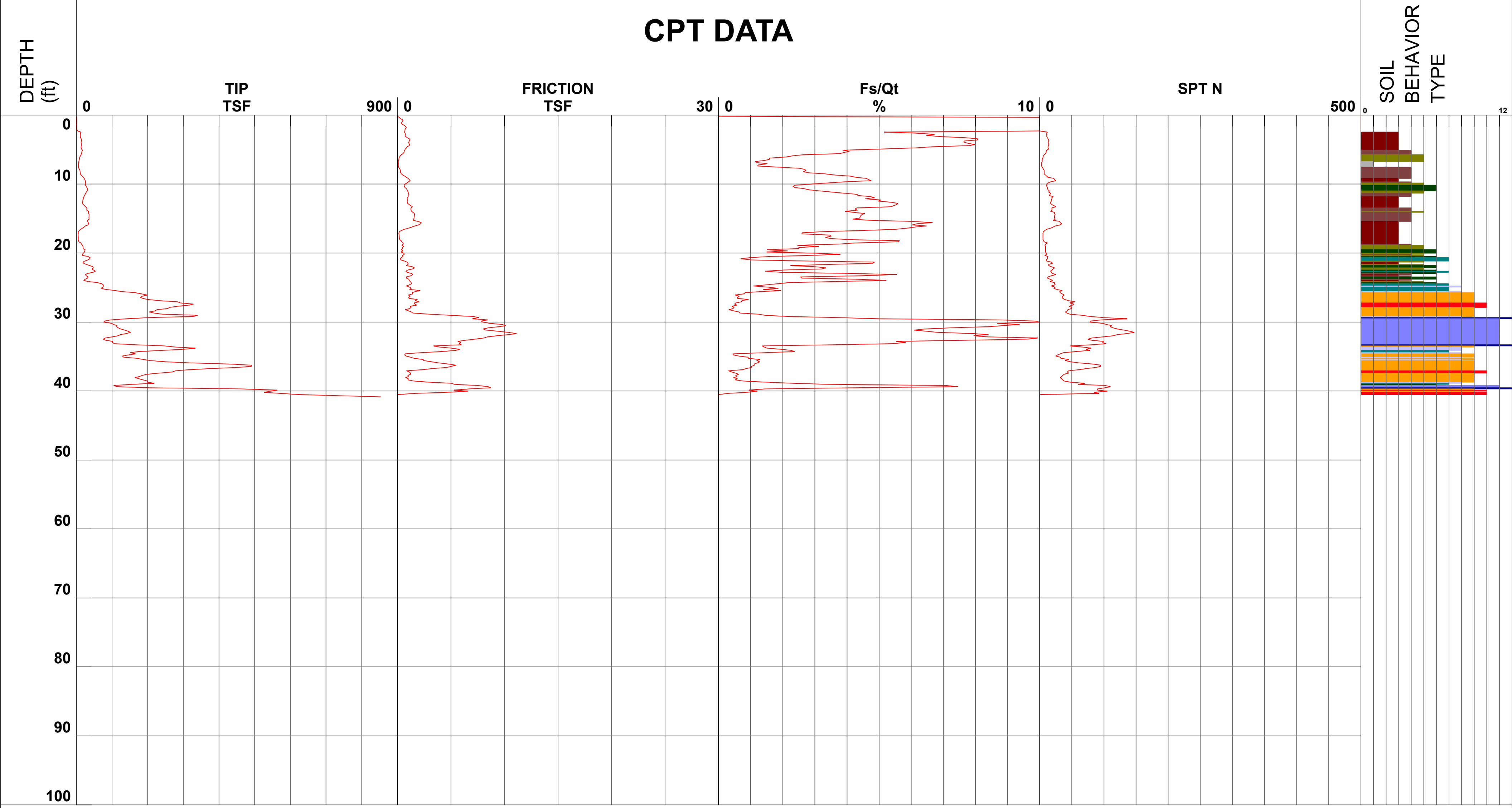
Wallace-Kuhl & Associates

Middle Earth Geo Testing Inc.

Project	Business Center Drive Apartments	Operator	JM-AJ-Olu	Filename	SDF(188).cpt
Job Number	13081.02	Cone Number	DDG1542	GPS	
Hole Number	CPT-01	Date and Time	3/17/2021 8:03:48 AM	Maximum Depth	40.85 ft
EST GW Depth During Test		3.00 ft			

Net Area Ratio .8

CPT DATA



- 1 - sensitive fine grained
- 2 - organic material
- 3 - clay
- 4 - silty clay to clay
- 5 - clayey silt to silty clay
- 6 - sandy silt to clayey silt
- 7 - silty sand to sandy silt
- 8 - sand to silty sand
- 9 - sand
- 10 - gravelly sand to sand
- 11 - very stiff fine grained (*)
- 12 - sand to clayey sand (*)

Cone Size 15cm squared

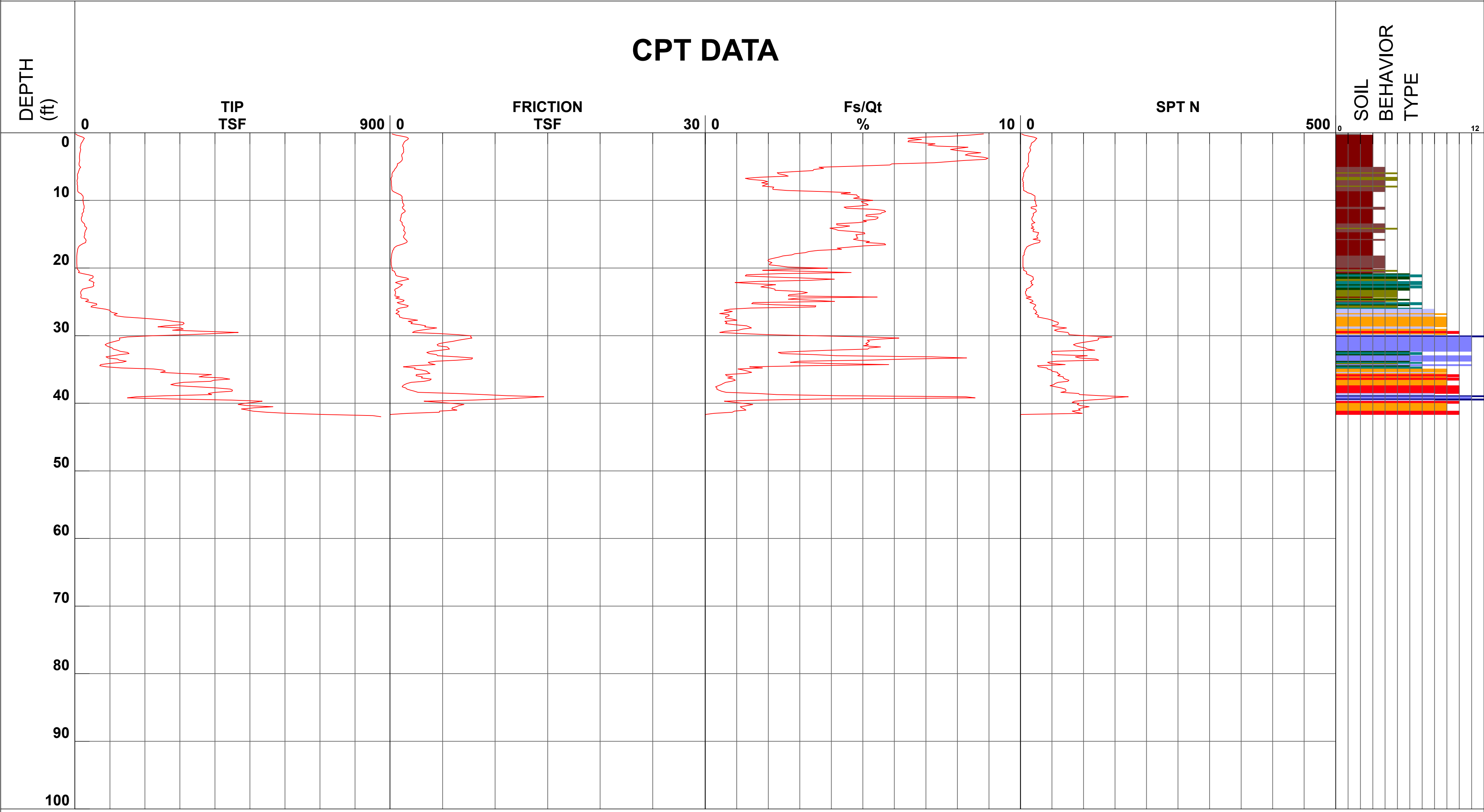
S*Soil behavior type and SPT based on data from UBC-1983

Wallace-Kuhl & Associates

Project	Business Center Drive Apartments	Operator	JM-AJ-Olu	Filename	SDF(189).cpt
Job Number	13081.02	Cone Number	DDG1542	GPS	
Hole Number	CPT-01A	Date and Time	3/17/2021 8:47:12 AM	Maximum Depth	41.99 ft
EST GW Depth During Test			3.00 ft		

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 15cm squared

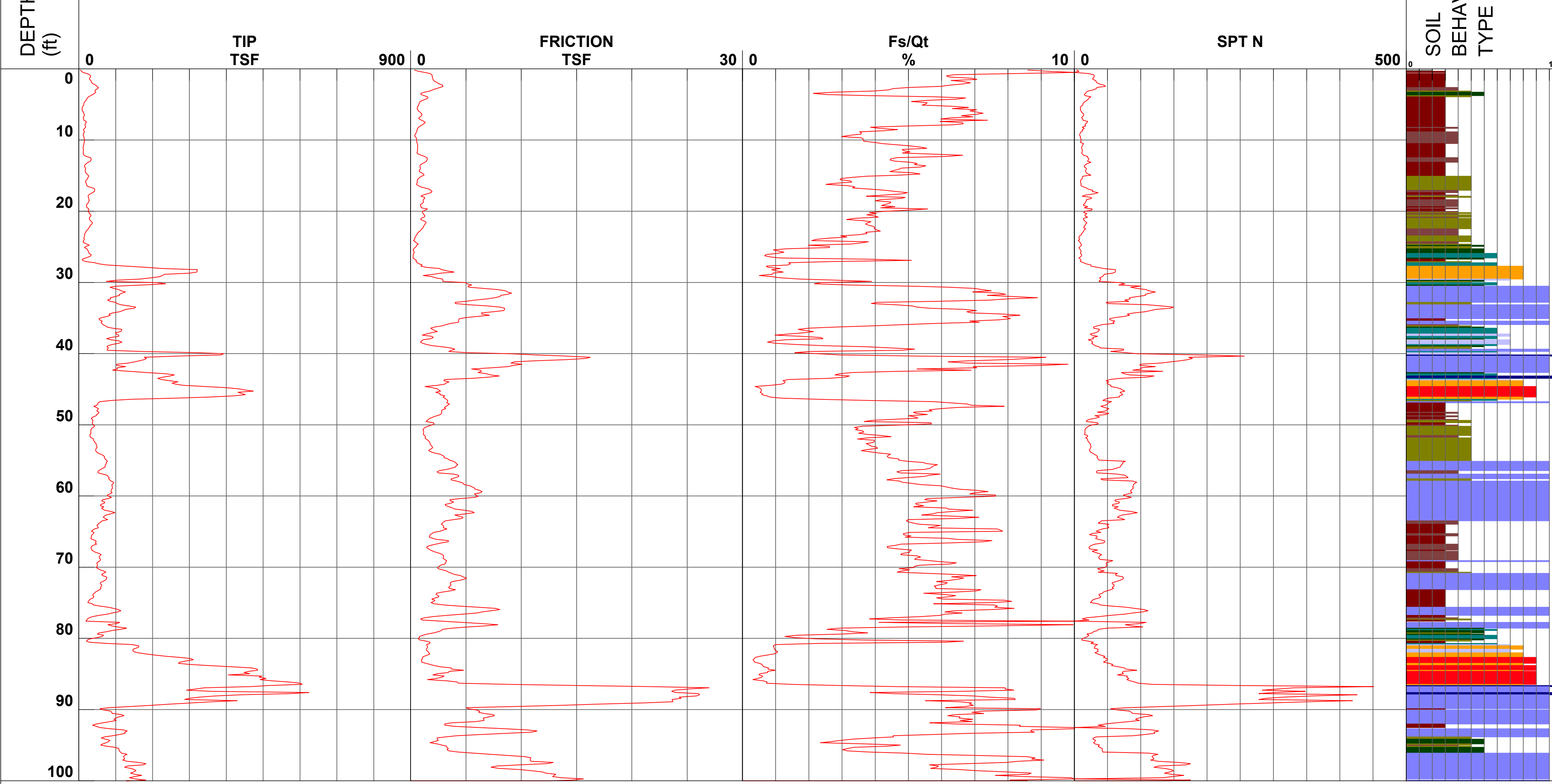
S*Soil behavior type and SPT based on data from UBC-1983

Wallace-Kuhl & Associates

Project	Business Center Drive Apartments	Operator	JM-AJ-Olu	Filename	SDF(190).cpt
Job Number	13081.02	Cone Number	DDG1542	GPS	
Hole Number	CPT-02	Date and Time	3/17/2021 9:38:02 AM	Maximum Depth	100.72 ft
EST GW Depth During Test			5.00 ft		

Net Area Ratio .8

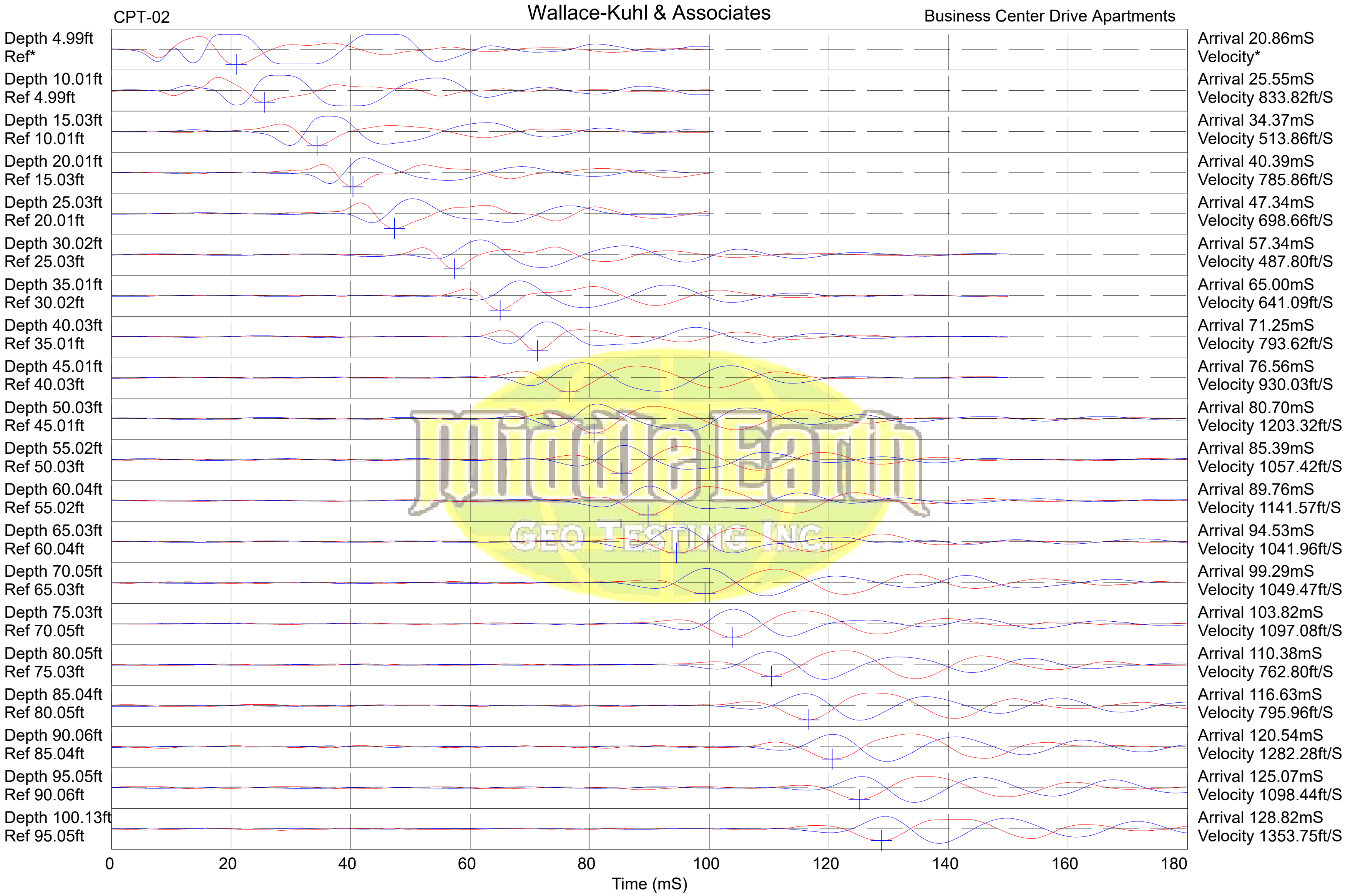
CPT DATA



- 1 - sensitive fine grained
- 2 - organic material
- 3 - clay
- 4 - silty clay to clay
- 5 - clayey silt to silty clay
- 6 - sandy silt to clayey silt
- 7 - silty sand to sandy silt
- 8 - sand to silty sand
- 9 - sand
- 10 - gravelly sand to sand
- 11 - very stiff fine grained (*)
- 12 - sand to clayey sand (*)

Cone Size 15cm squared

S*Soil behavior type and SPT based on data from UBC-1983



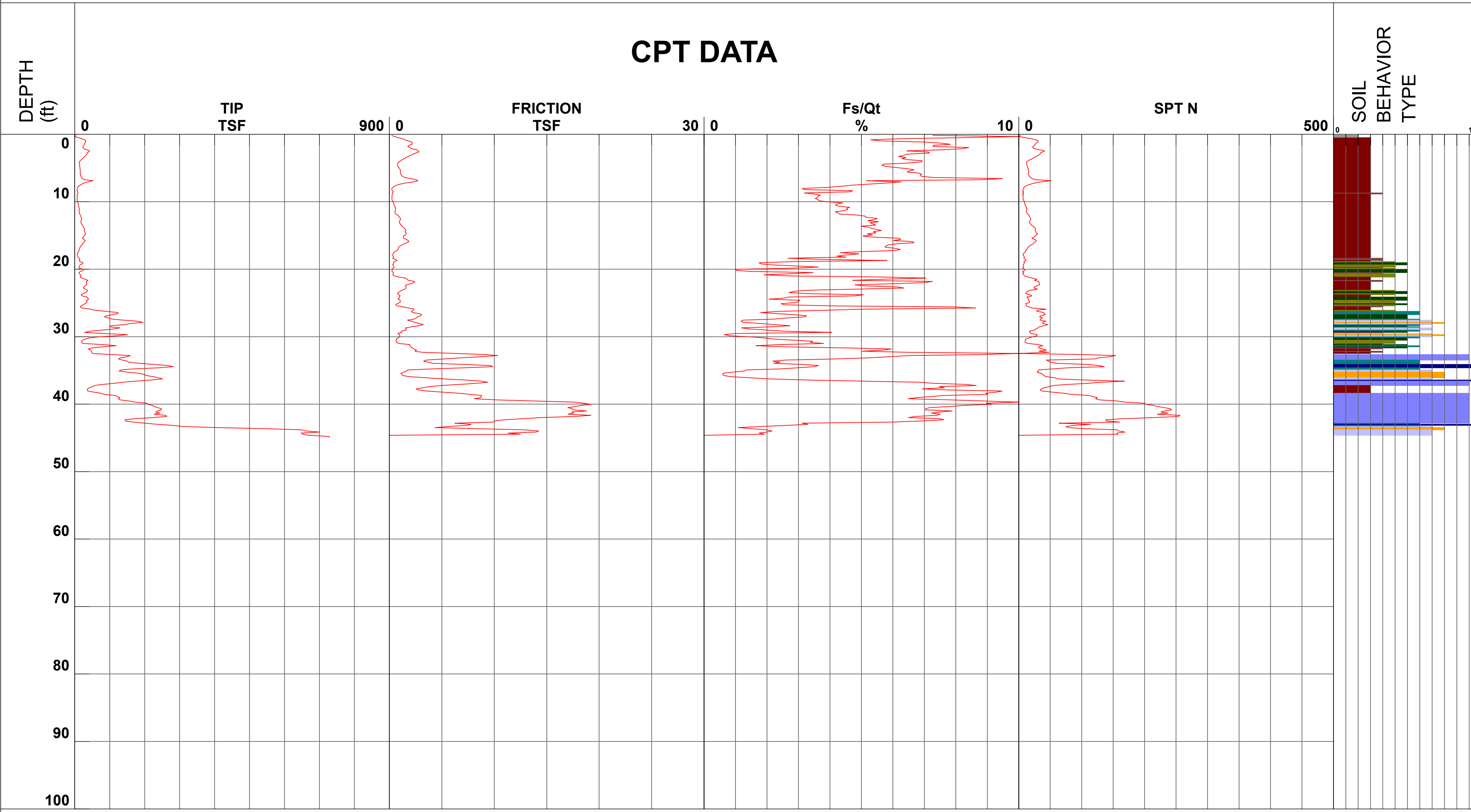
Hammer to Rod String Distance (ft): 5.83
* = Not Determined

COMMENT:

Wallace-Kuhl & Associates

Project	Business Center Drive Apartments	Operator	JM-AJ-Olu	Filename	SDF(191).cpt
Job Number	13081.02	Cone Number	DDG1542	GPS	
Hole Number	CPT-03	Date and Time	3/17/2021 11:06:23 AM	Maximum Depth	44.95 ft
EST GW Depth During Test		5.00 ft			

Net Area Ratio .8



- 1 - sensitive fine grained
- 2 - organic material
- 3 - clay
- 4 - silty clay to clay
- 5 - clayey silt to silty clay
- 6 - sandy silt to clayey silt
- 7 - silty sand to sandy silt
- 8 - sand to silty sand
- 9 - sand
- 10 - gravelly sand to sand
- 11 - very stiff fine grained (*)
- 12 - sand to clayey sand (*)

Cone Size 15cm squared

S*Soil behavior type and SPT based on data from UBC-1983

APPENDIX C
Liquefaction Analysis Output Files

Wallace-Kuhl & Associates

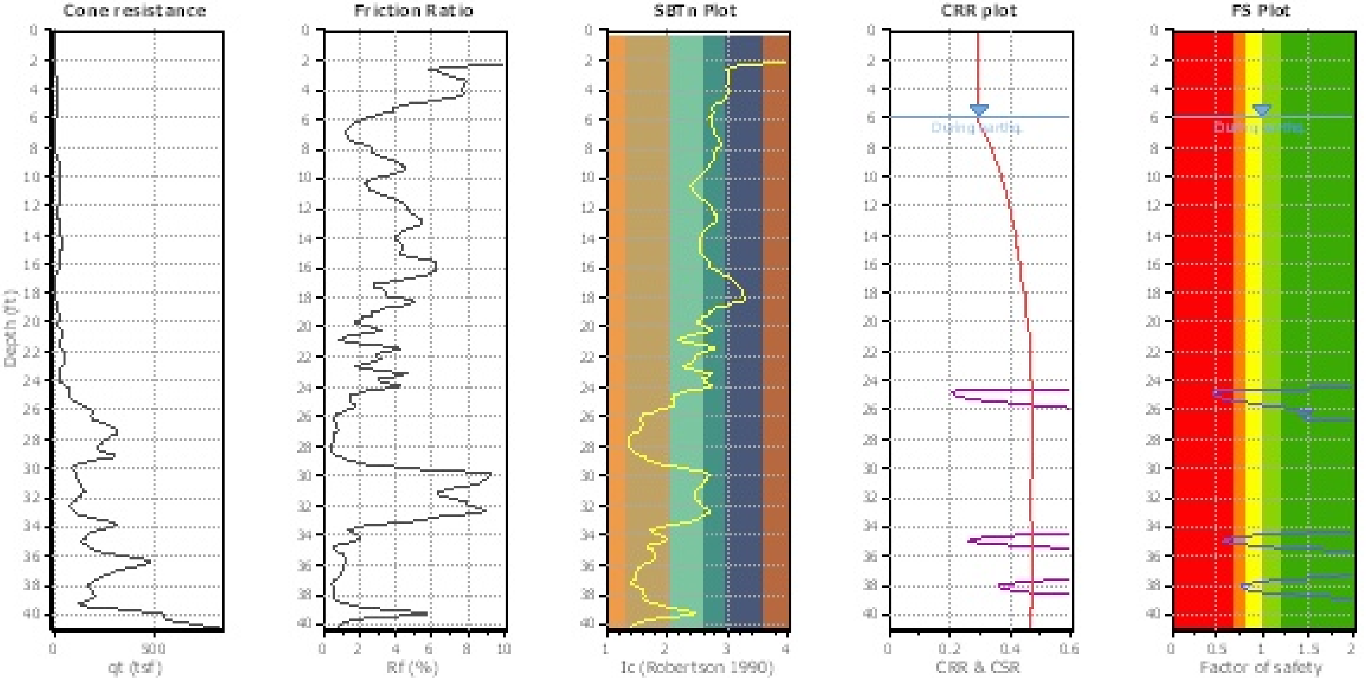
LIQUEFACTION ANALYSIS REPORT

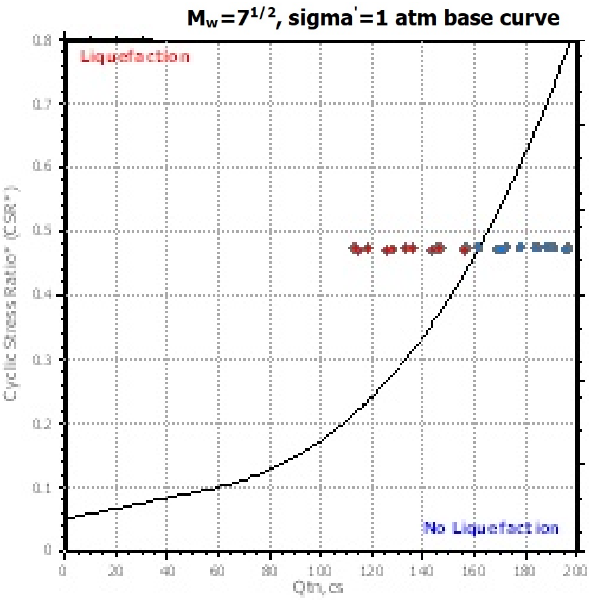
Project title : 13081.02 - Business Center Drive Apartments **Location : Fairfield**

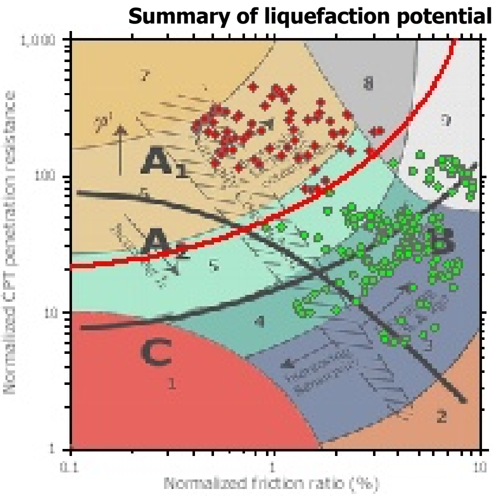
CPT file : CPT-01

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	8.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	6.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	60.00 ft
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.15	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.60	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

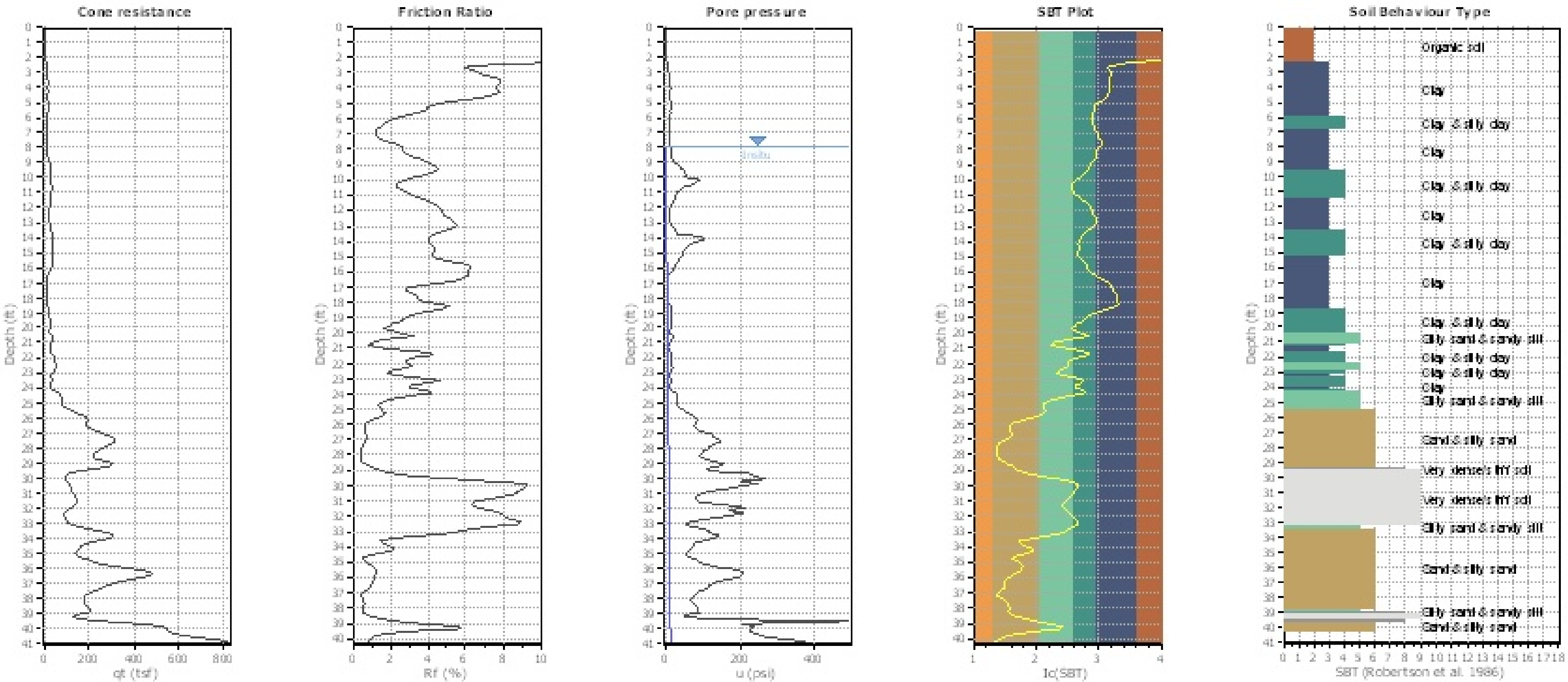






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plo



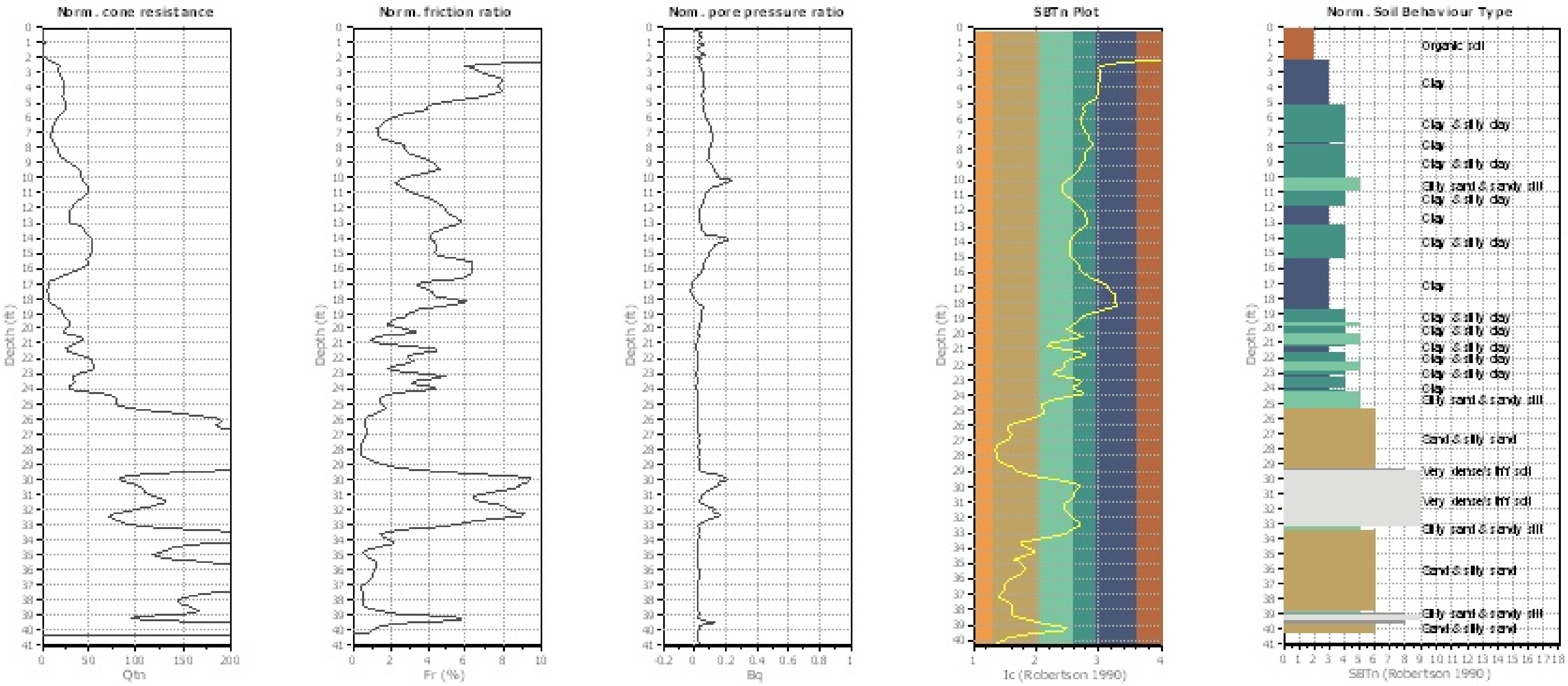
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	6.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.15	K_σ applied:	Yes
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

SBT legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to
- 9. Very stiff fine grained

CPT basic interpretation plots (normaliz



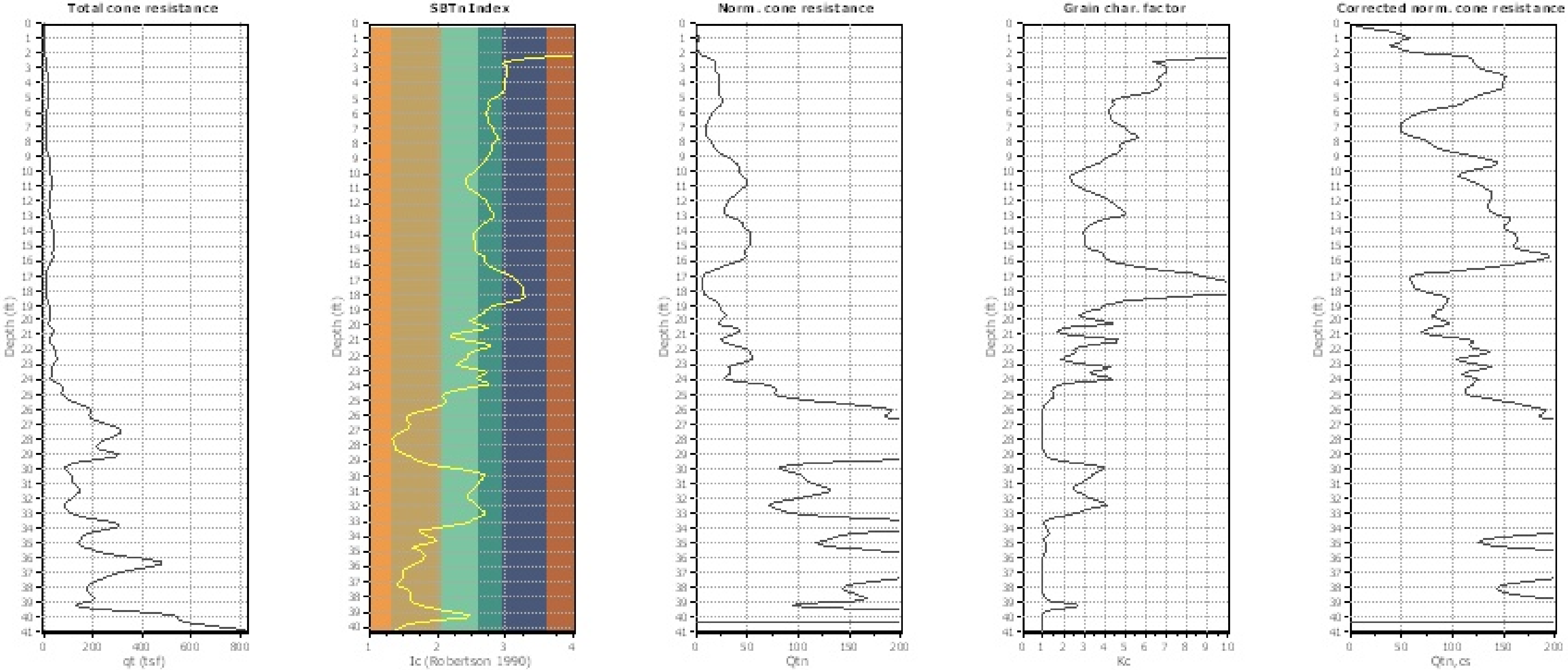
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	6.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.15	K_σ applied:	Yes
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

SBTn legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to
- 9. Very stiff fine grained

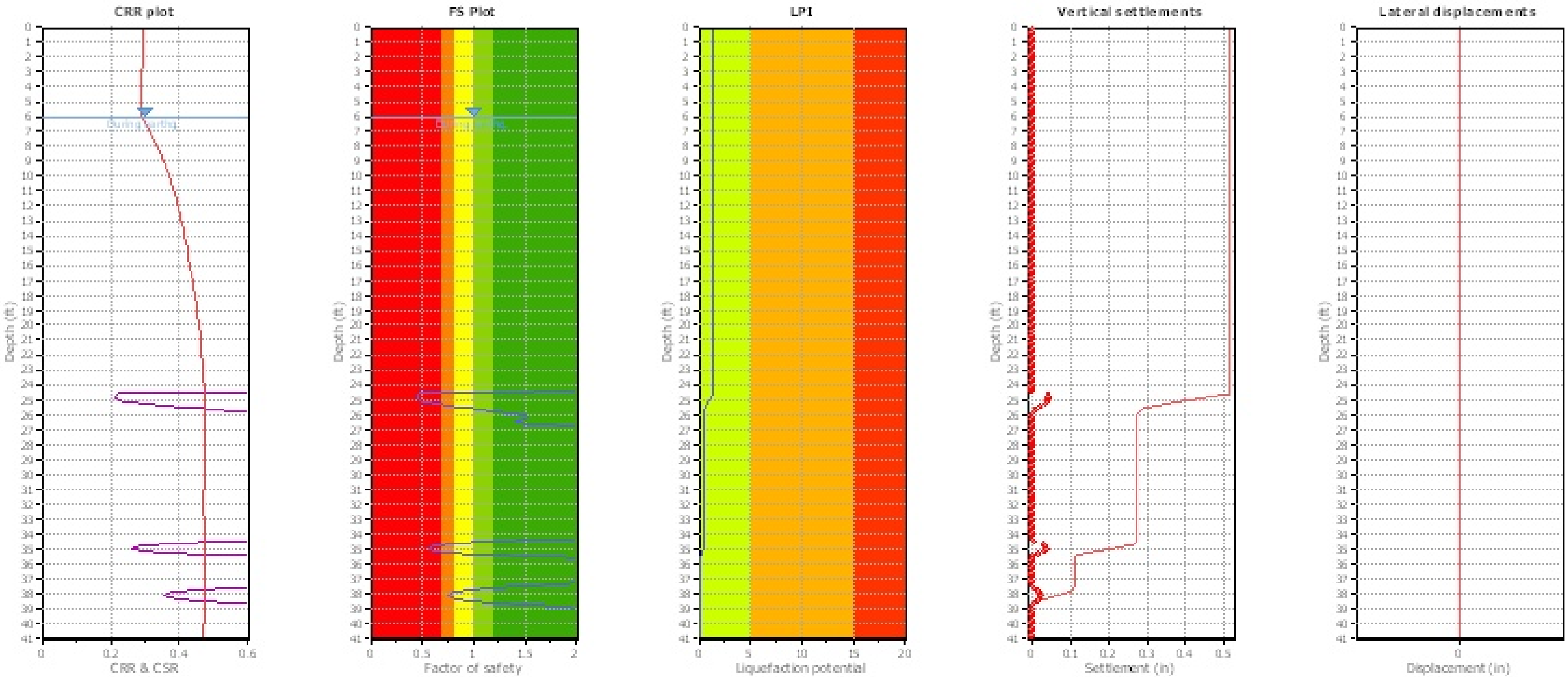
Liquefaction analysis overall plots (intermediate resu



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	6.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.15	K_σ applied:	Yes
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	6.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.15	K_σ applied:	Yes
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

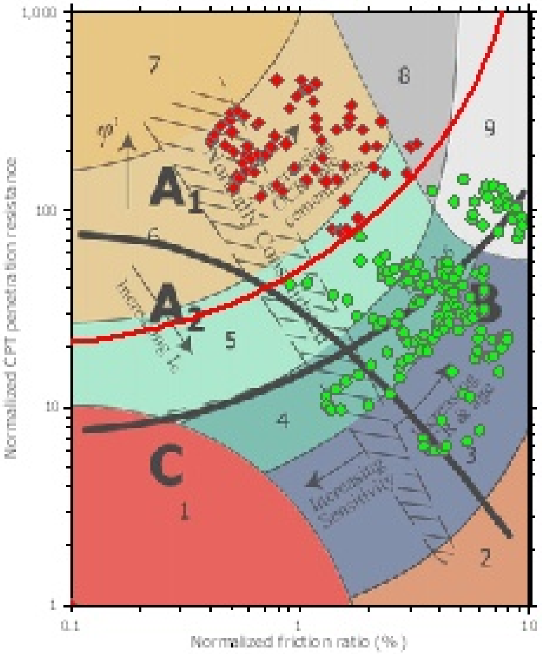
F.S. color scheme

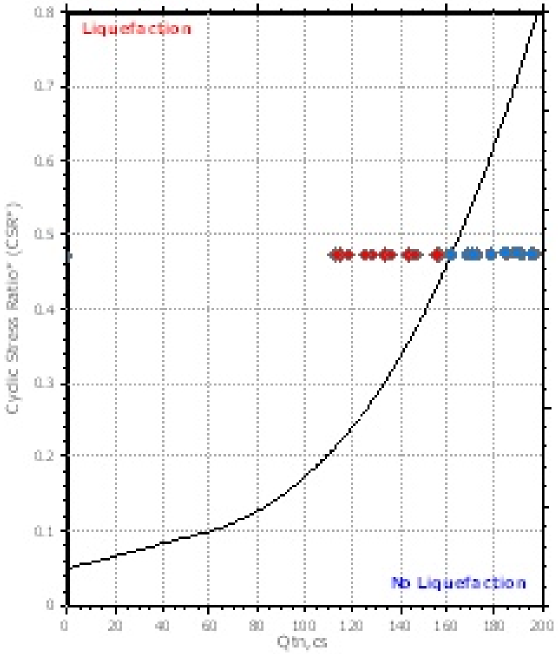
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

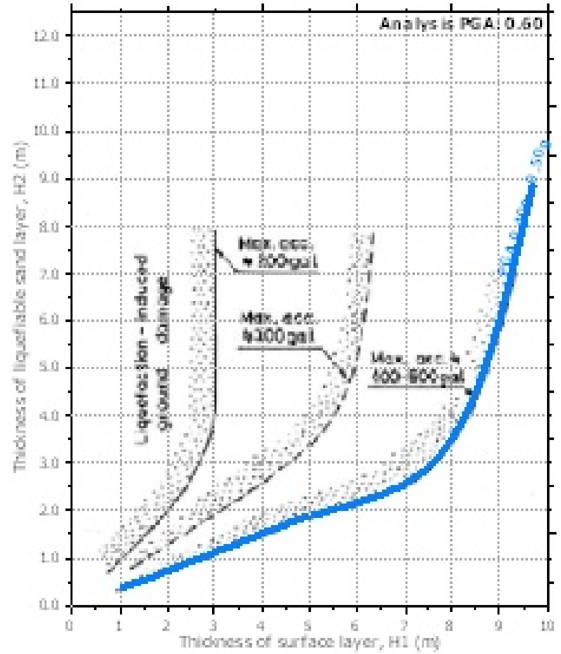
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plo



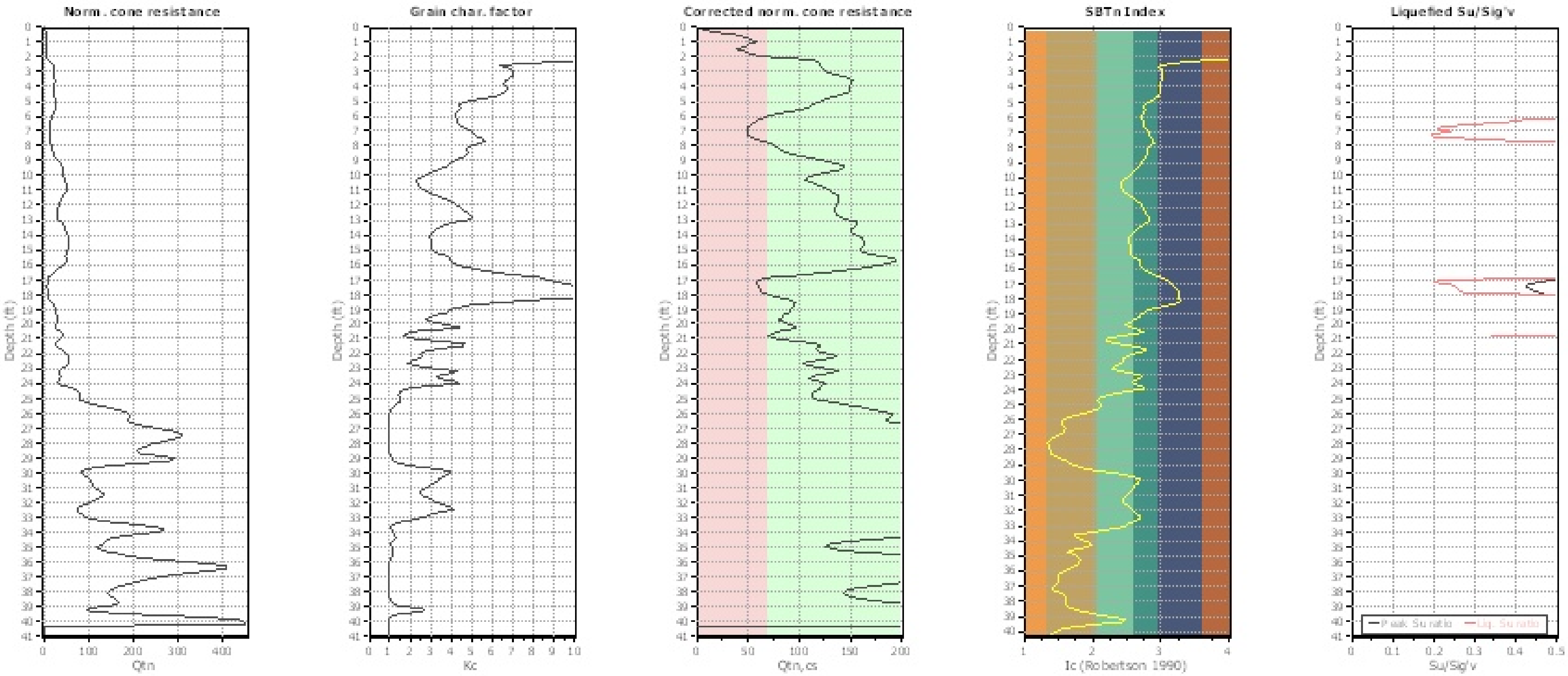




Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	6.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.15	K_σ applied:	Yes
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

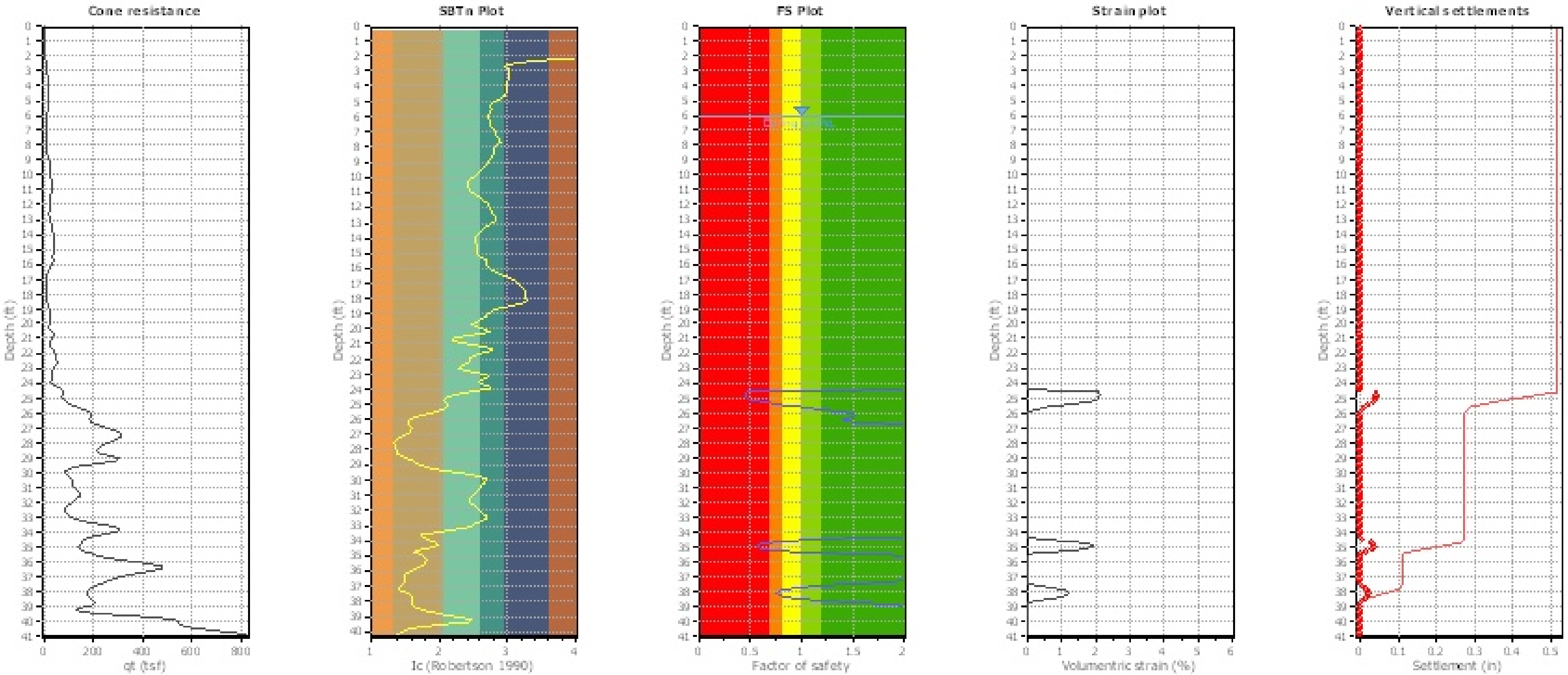
Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	6.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.15	K_σ applied:	Yes
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Wallace-Kuhl & Associates

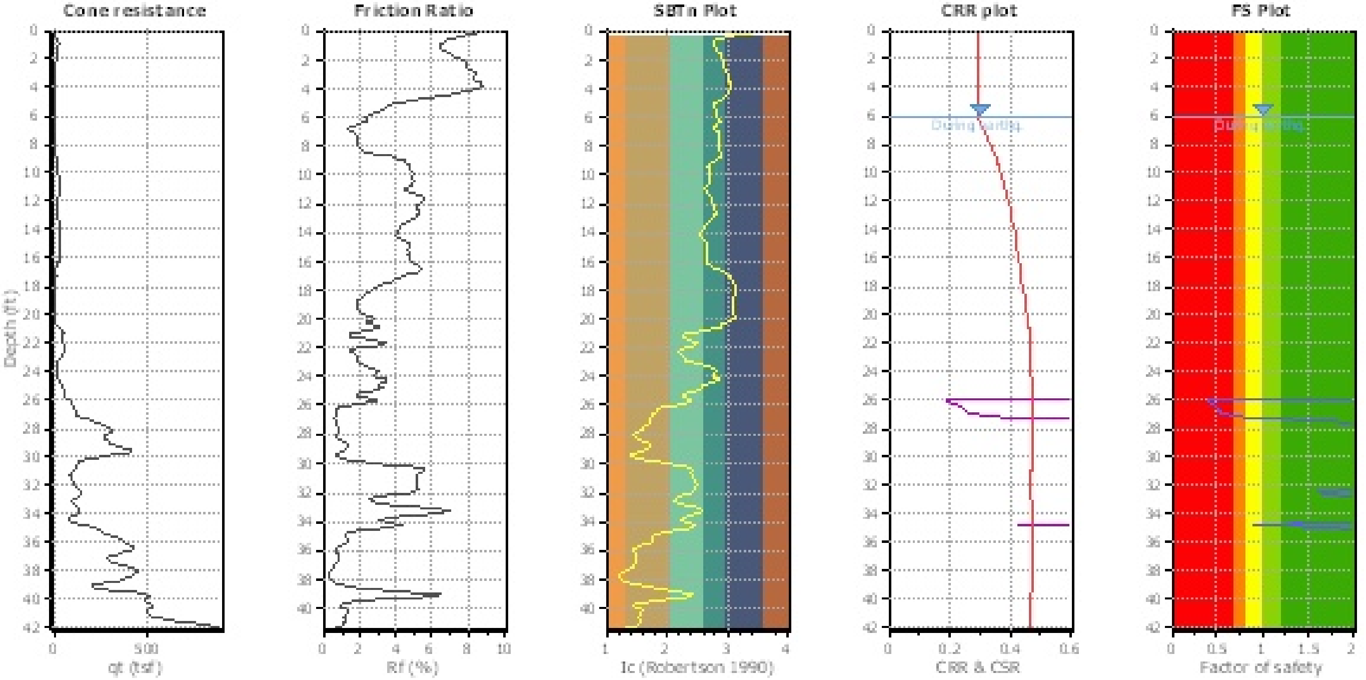
LIQUEFACTION ANALYSIS REPORT

Project title : 13081.02 - Business Center Drive Apartments **Location : Fairfield**

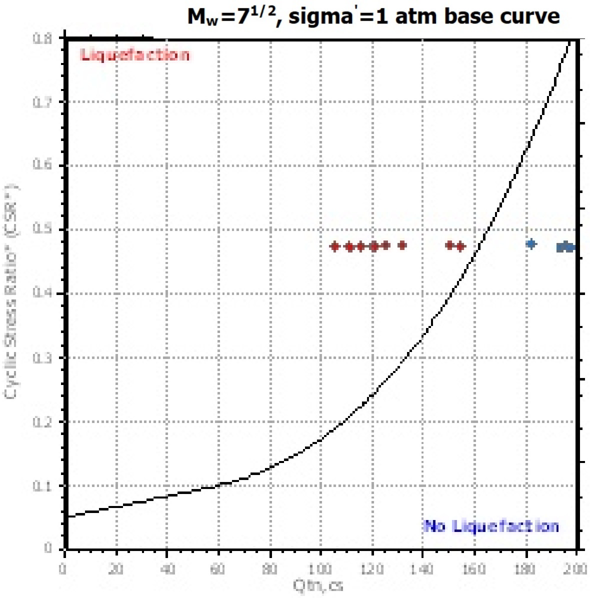
CPT file : CPT-01A

Input parameters and analysis data

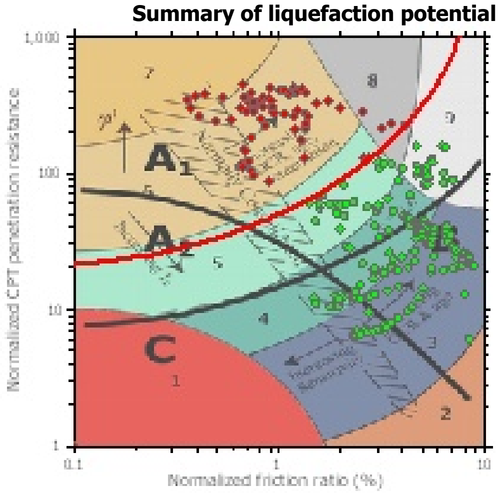
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	8.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	6.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	60.00 ft
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.15	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.60	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



$M_w=7^{1/2}$, sigma'=1 atm base curve

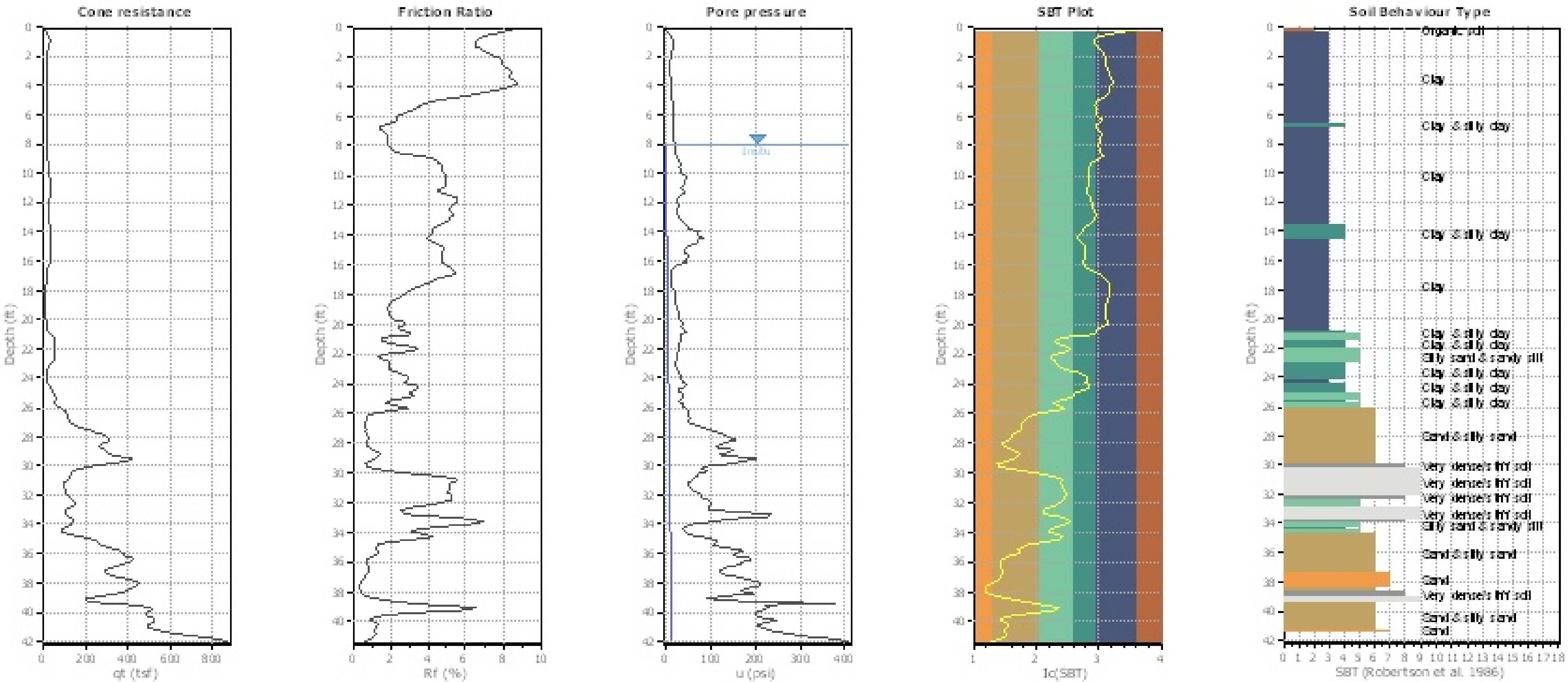


Summary of liquefaction potential



Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plo



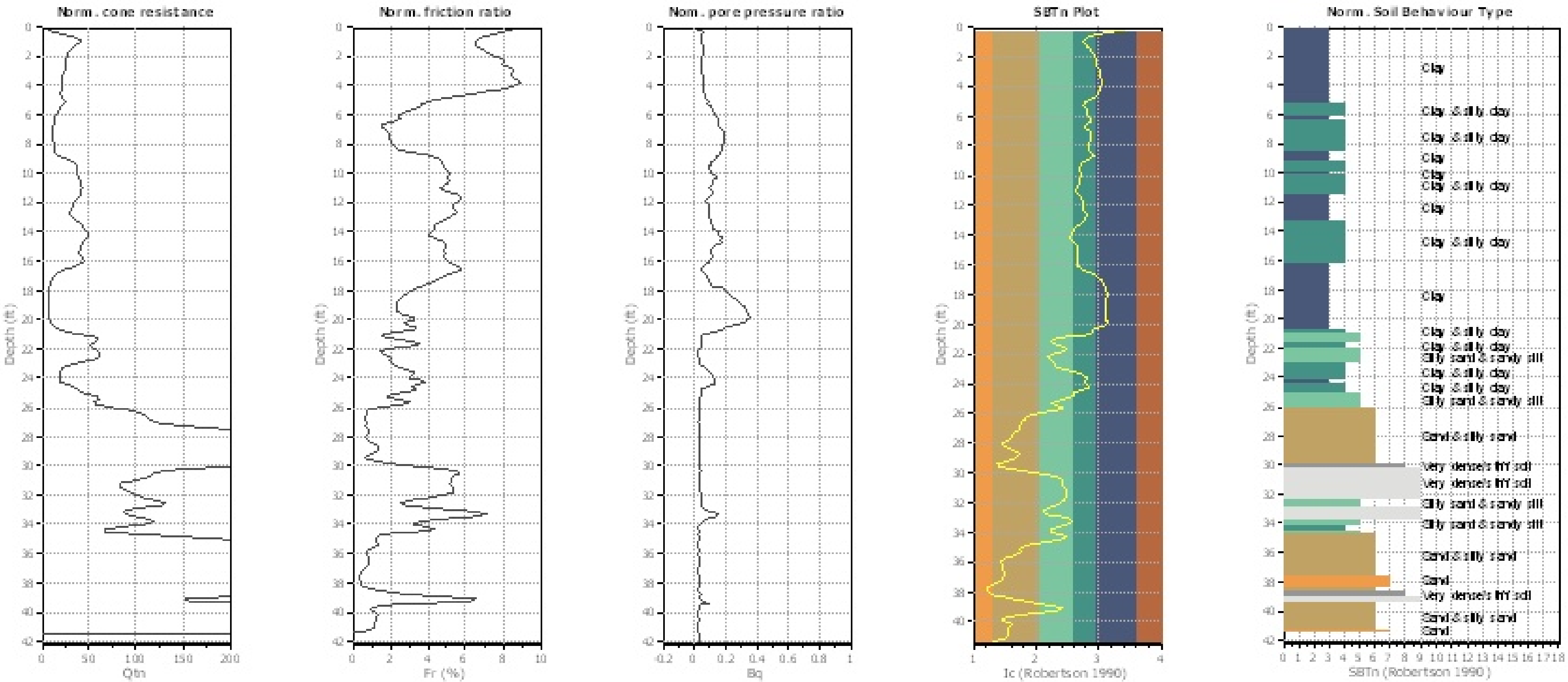
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	6.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.15	K_σ applied:	Yes
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty clay	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to clayey sand
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

CPT basic interpretation plots (normaliz



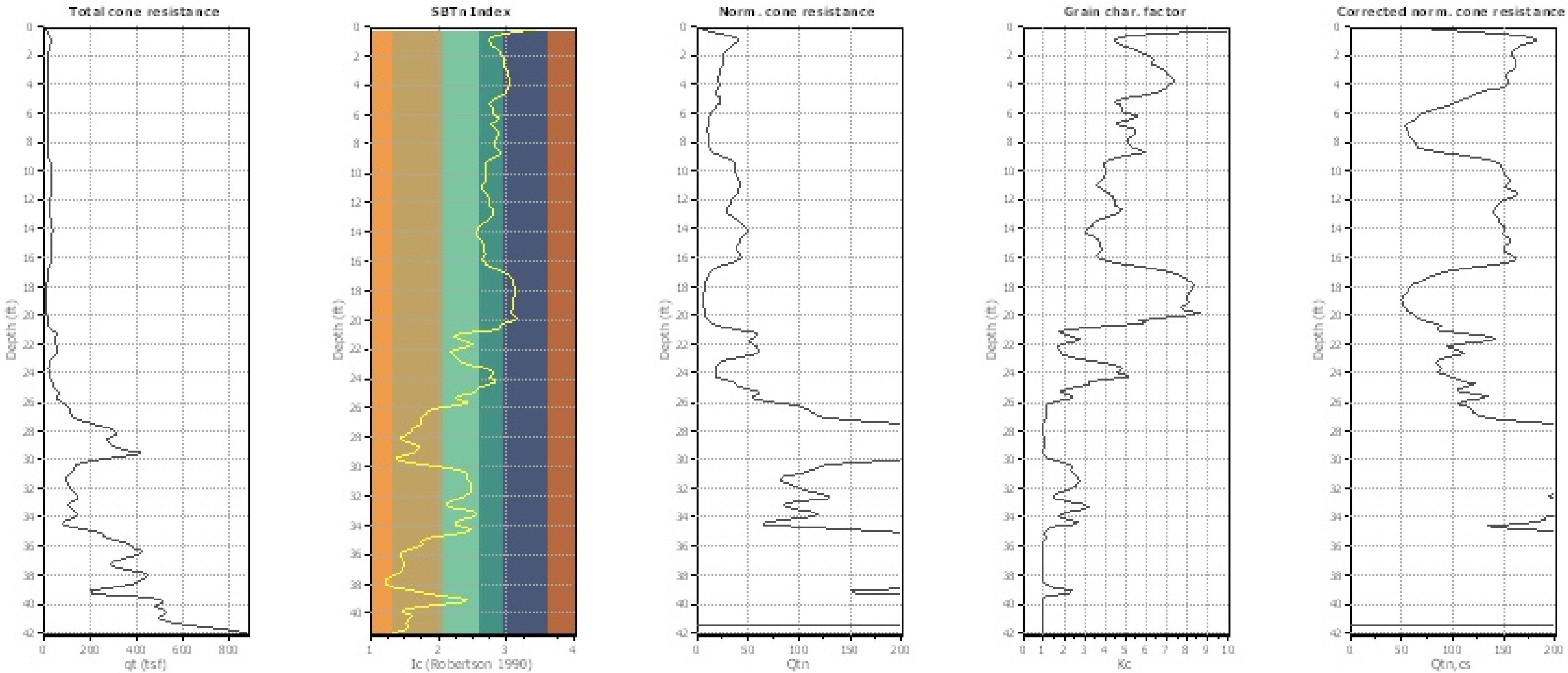
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	6.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.15	K_σ applied:	Yes
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

SBTn legend

1. Sensitive fine grained	4. Clayey silt to silty clay	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to clayey sand
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

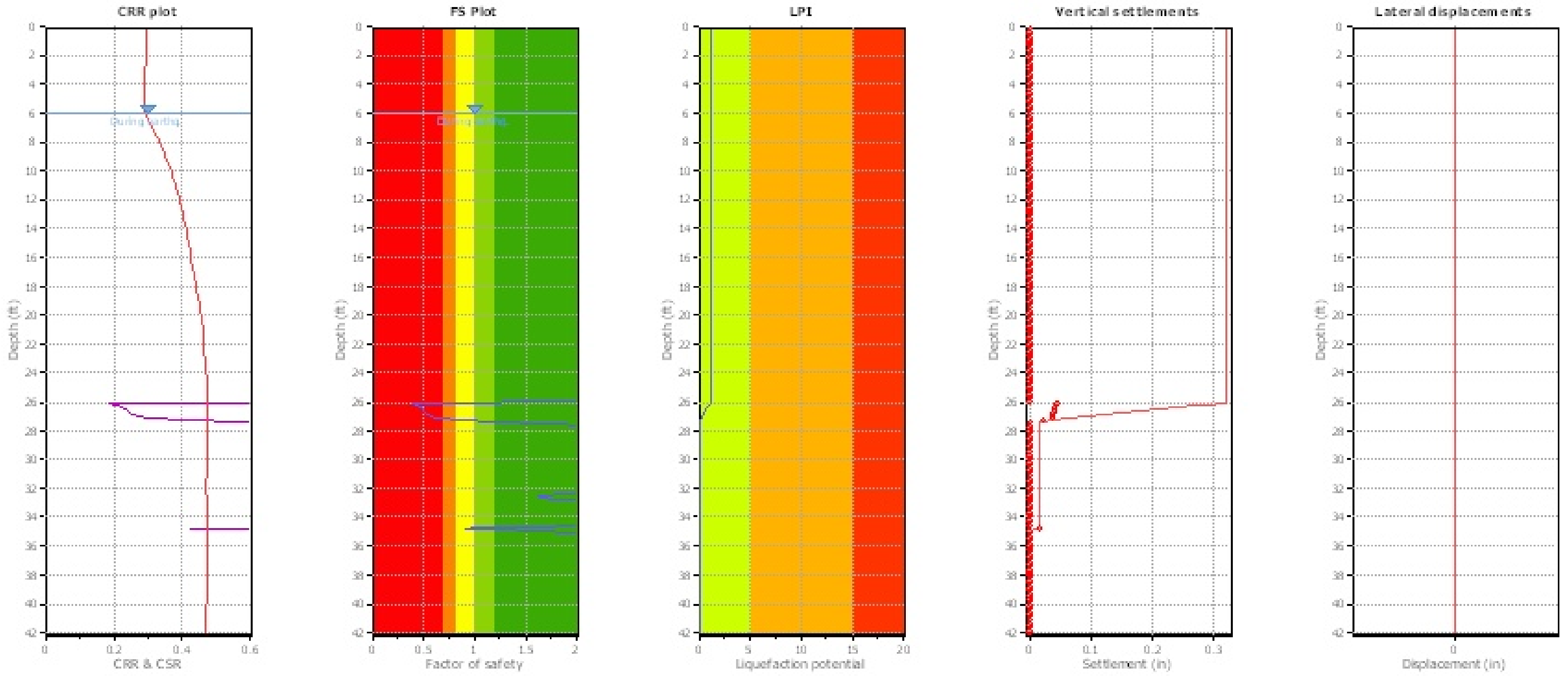
Liquefaction analysis overall plots (intermediate resu



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	6.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.15	K_σ applied:	Yes
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	6.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.15	K_σ applied:	Yes
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

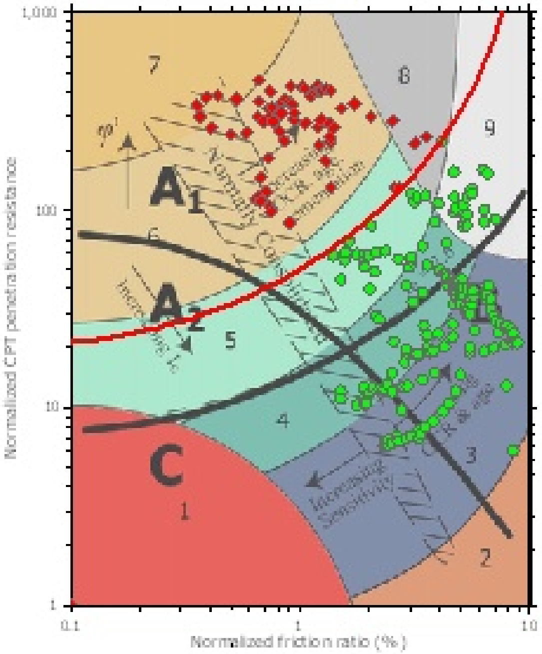
F.S. color scheme

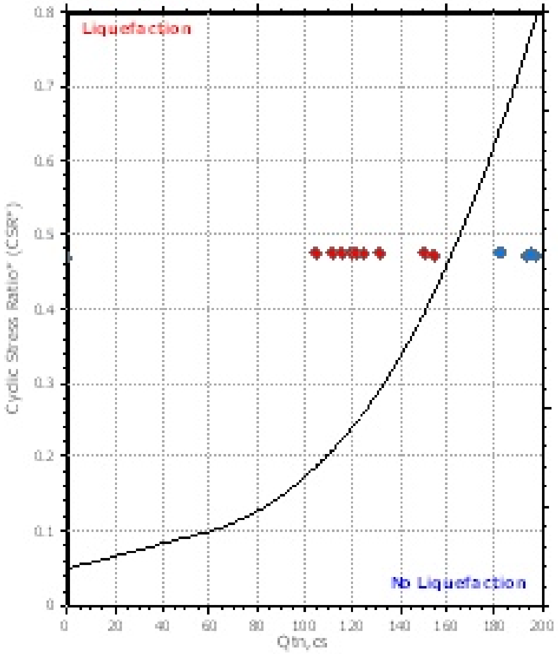
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

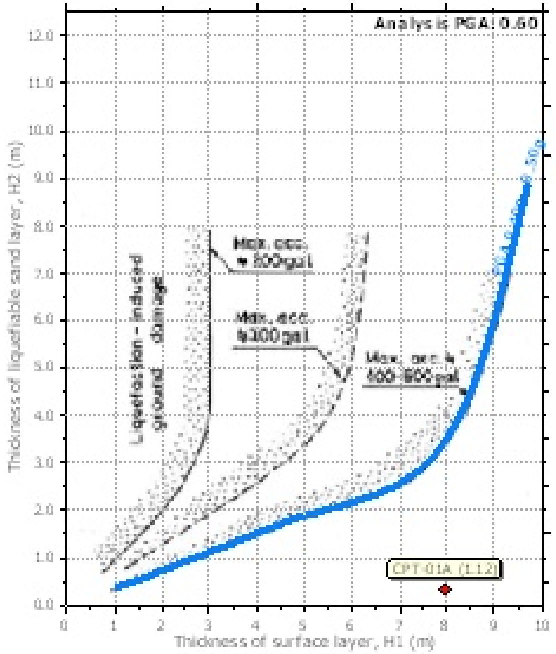
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plo



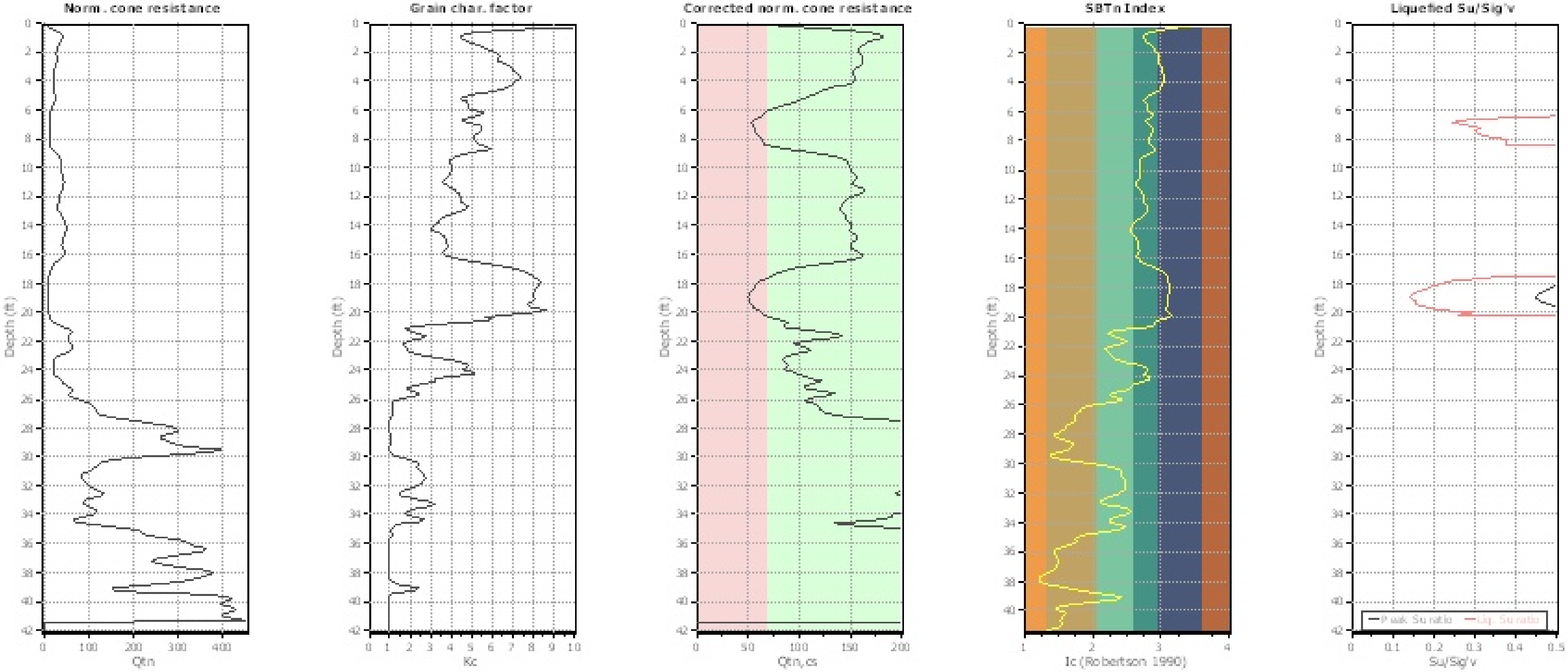




Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	6.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.15	K_σ applied:	Yes
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

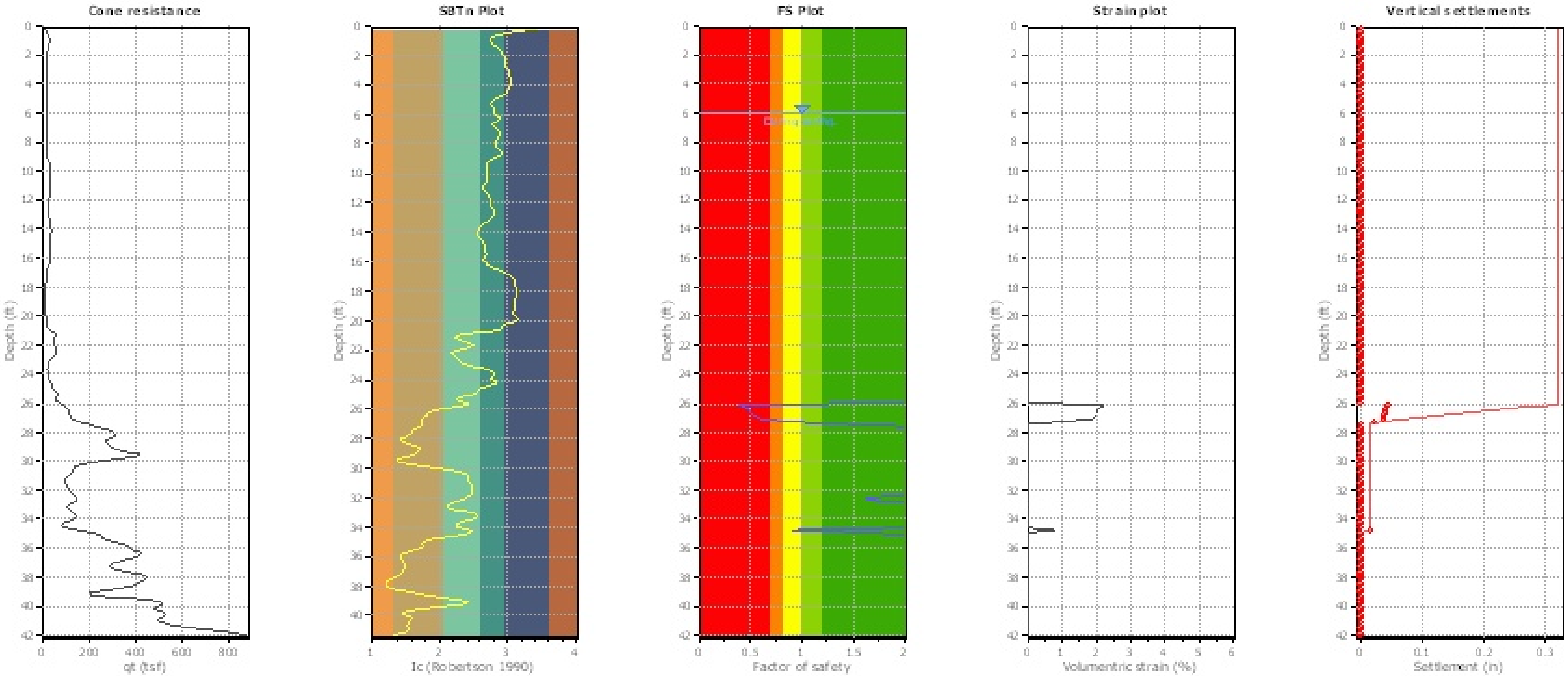
Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	6.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.15	K_σ applied:	Yes
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

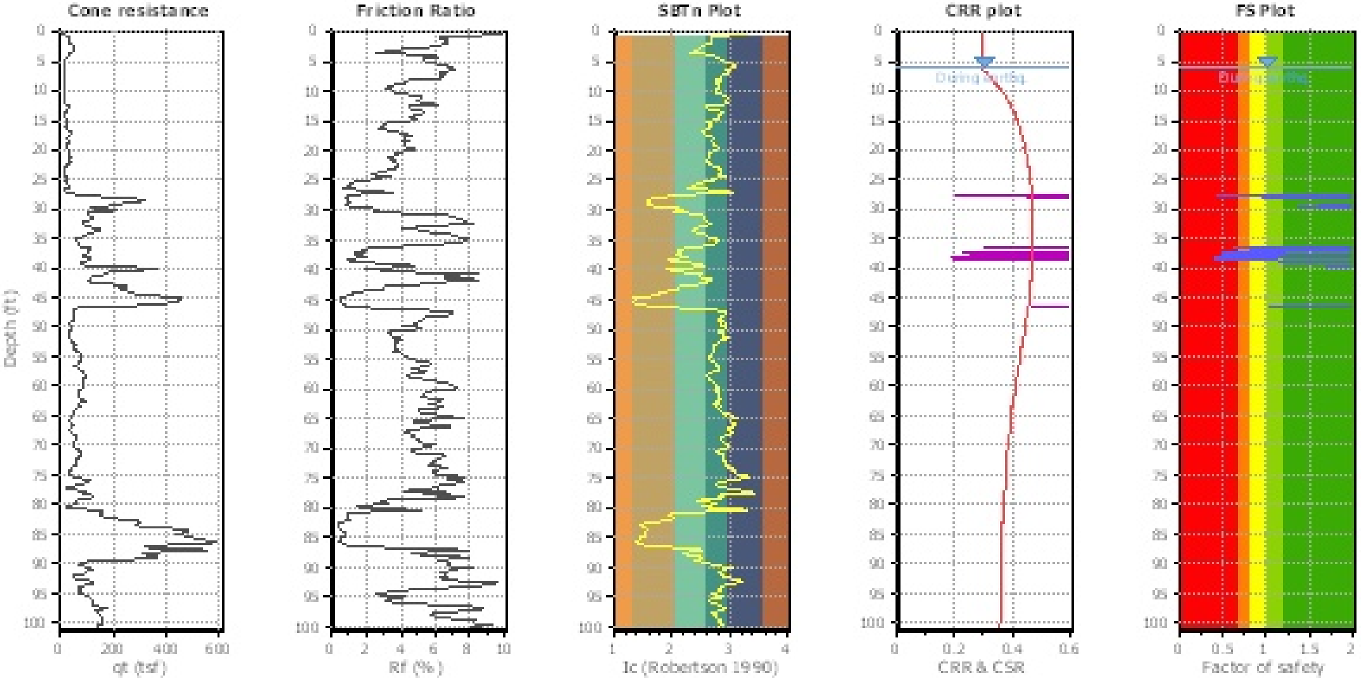
LIQUEFACTION ANALYSIS REPORT

Project title : 13081.02 - Business Center Drive Apartments **Location : Fairfield**

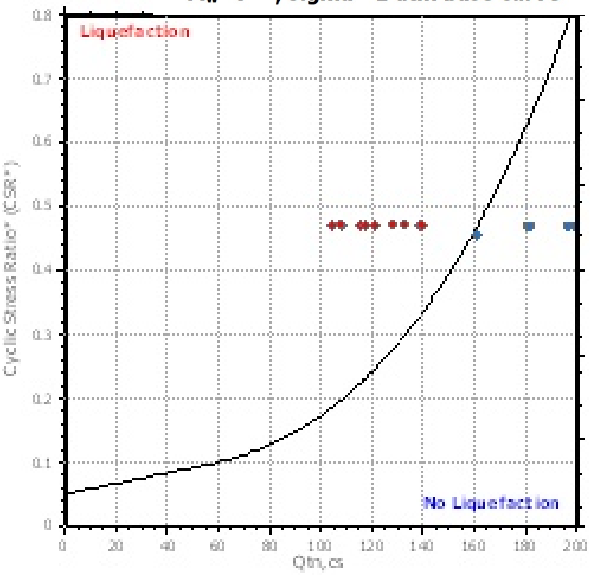
CPT file : CPT-02

Input parameters and analysis data

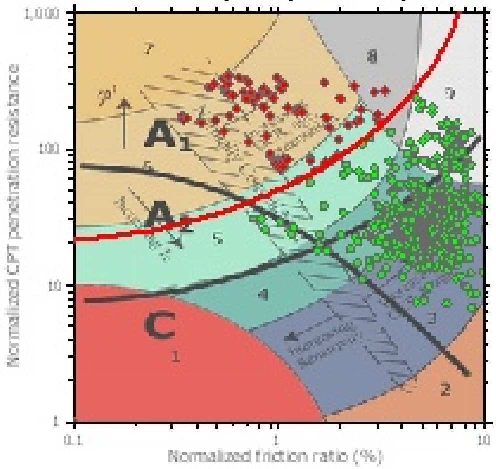
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	8.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	6.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	60.00 ft
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.15	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.60	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



$M_w=7^{1/2}$, sigma'=1 atm base curve

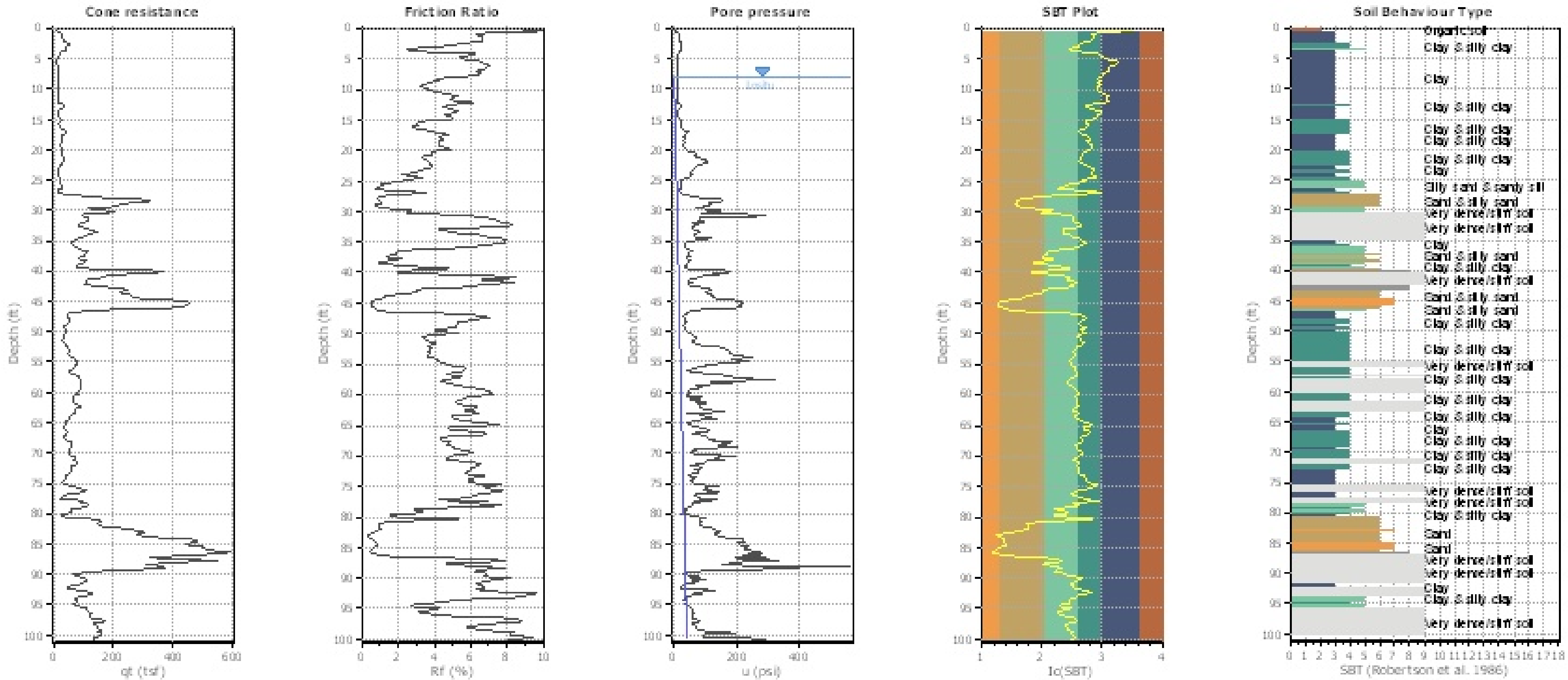


Summary of liquefaction potential



Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plo



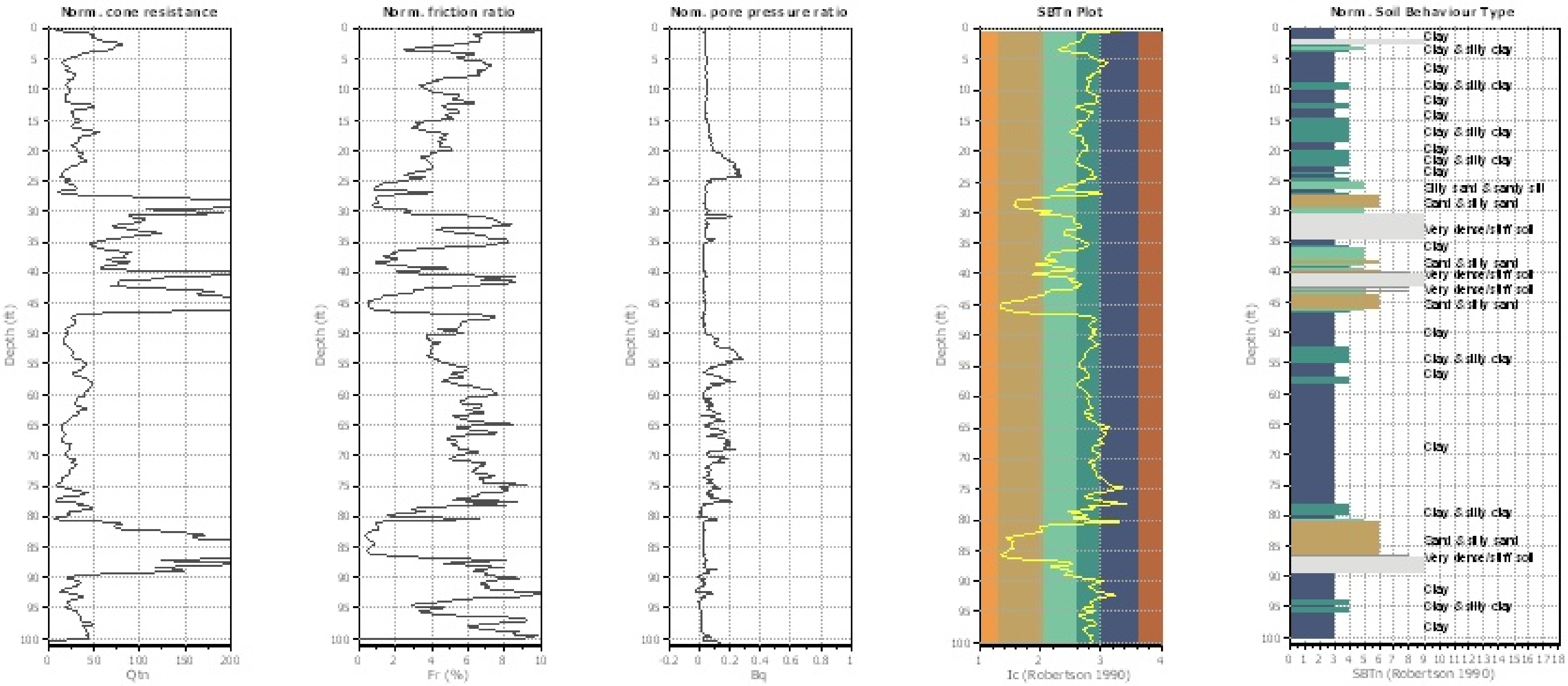
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	6.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.15	K_σ applied:	Yes
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

CPT basic interpretation plots (normaliz



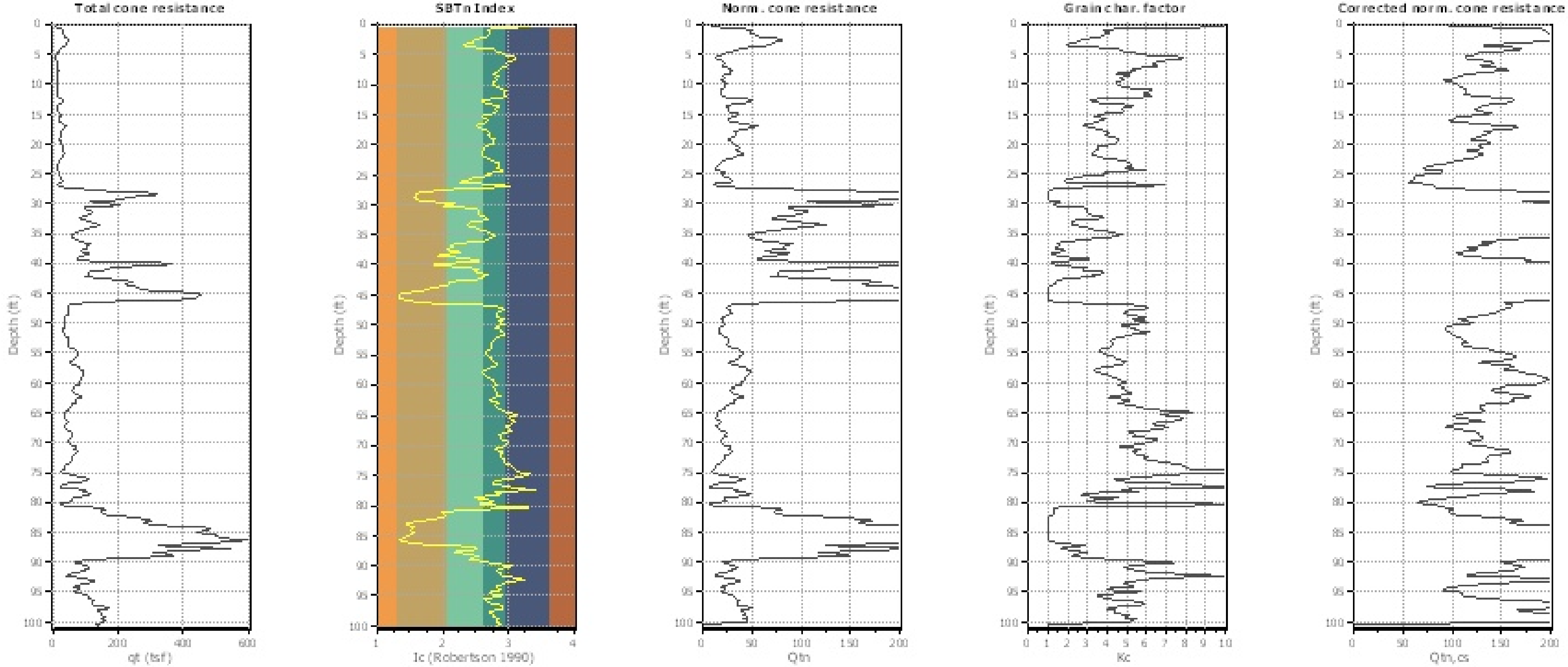
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	6.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.15	K_σ applied:	Yes
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

SBTn legend

1. Sensitive fine grained	4. Clayey silt to silty clay	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to clayey sand
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

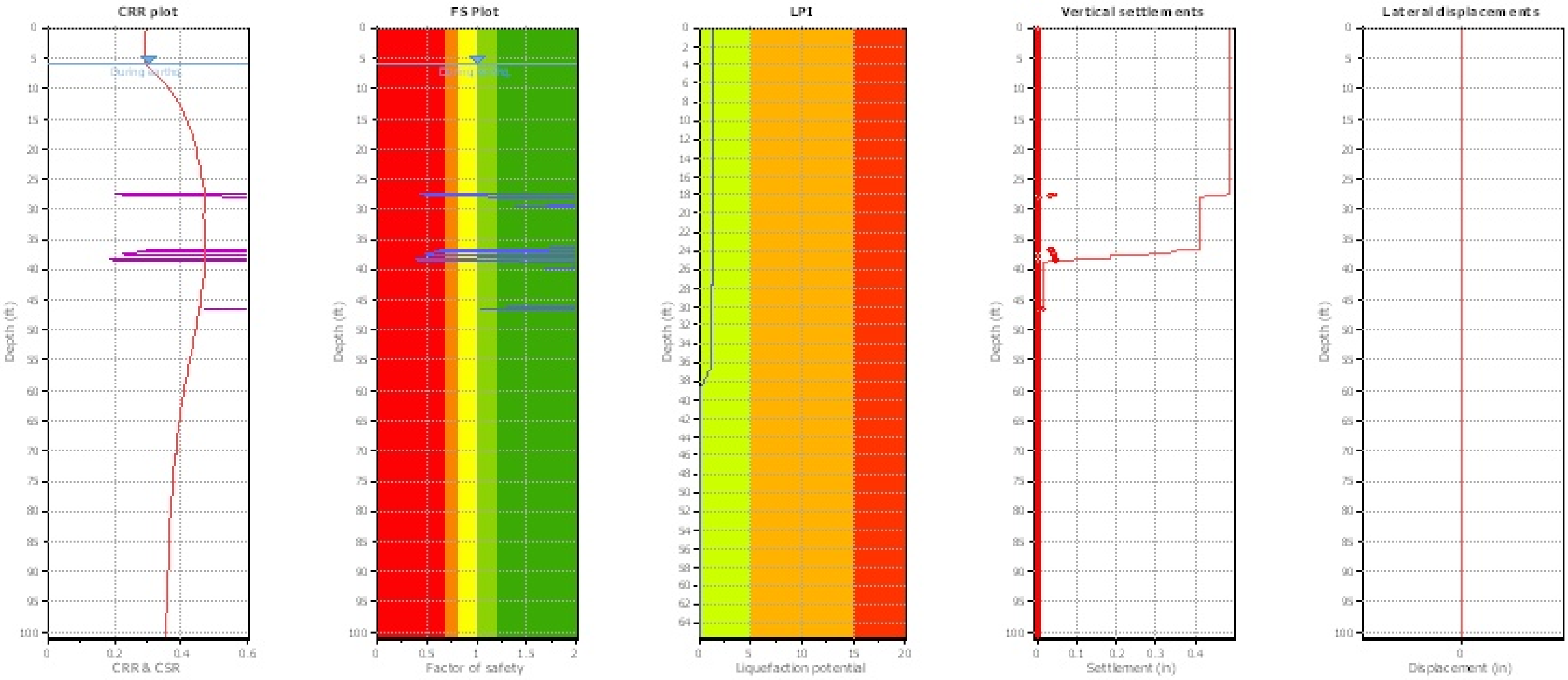
Liquefaction analysis overall plots (intermediate resu



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	6.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.15	K_σ applied:	Yes
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	6.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.15	K_σ applied:	Yes
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

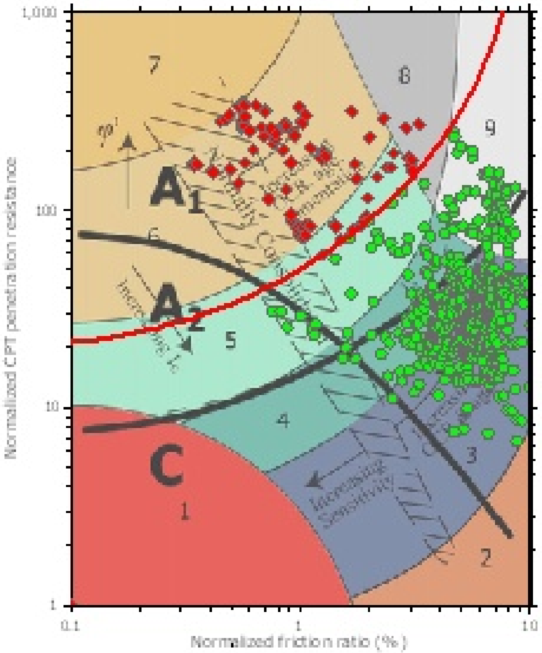
F.S. color scheme

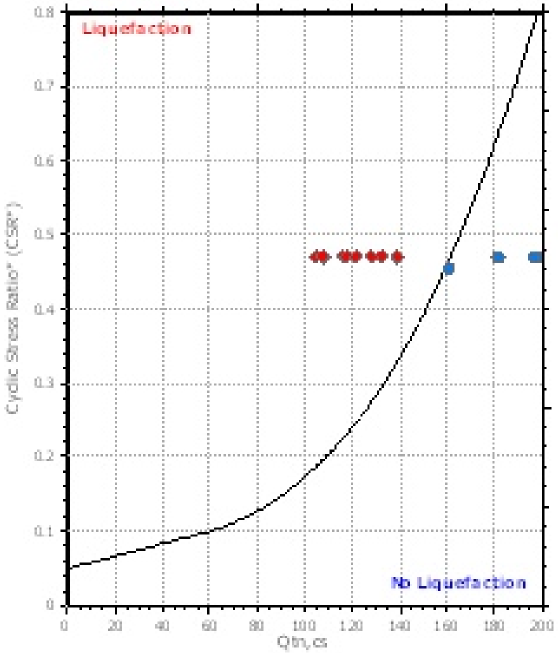
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

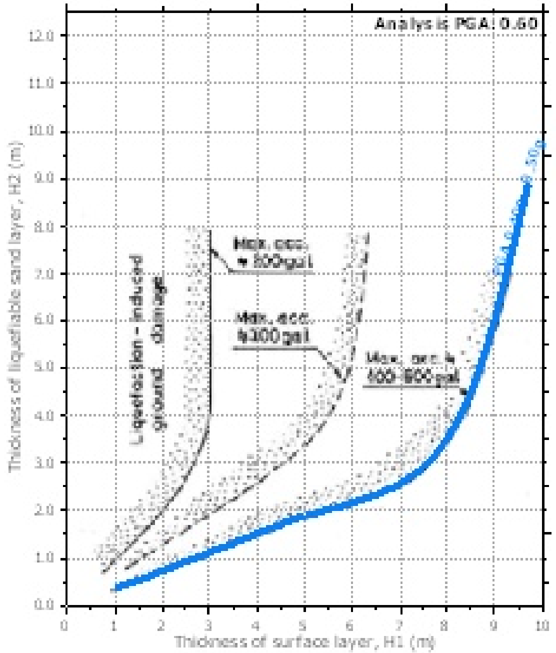
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plo



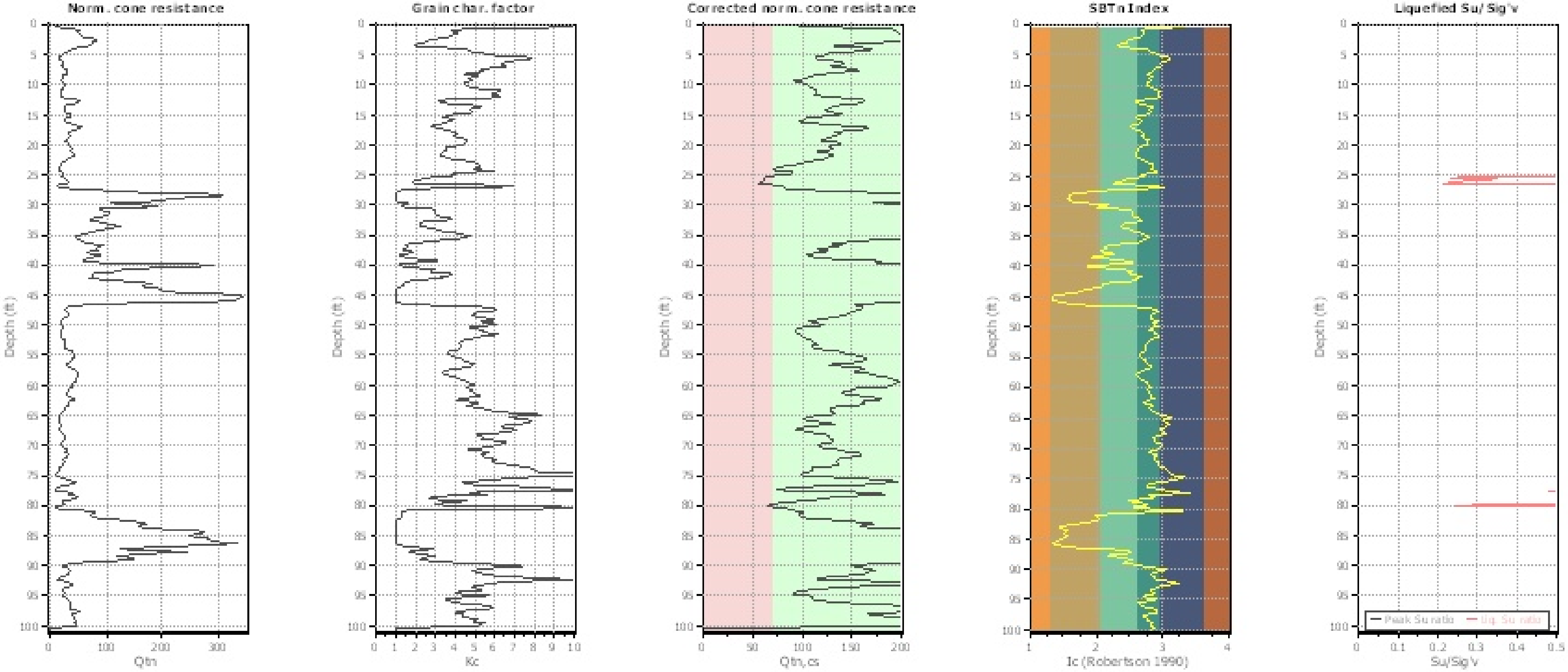




Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	6.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.15	K_σ applied:	Yes
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

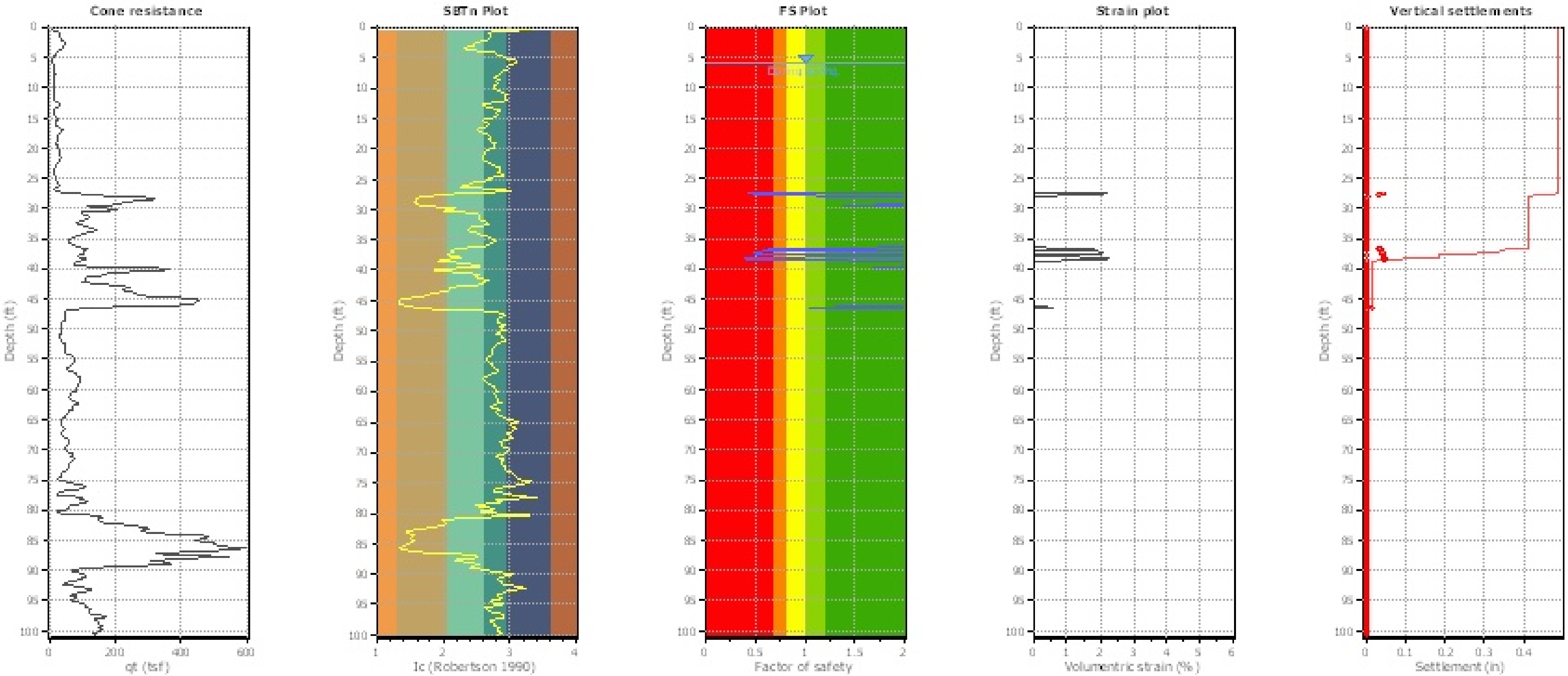
Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	6.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.15	K_σ applied:	Yes
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

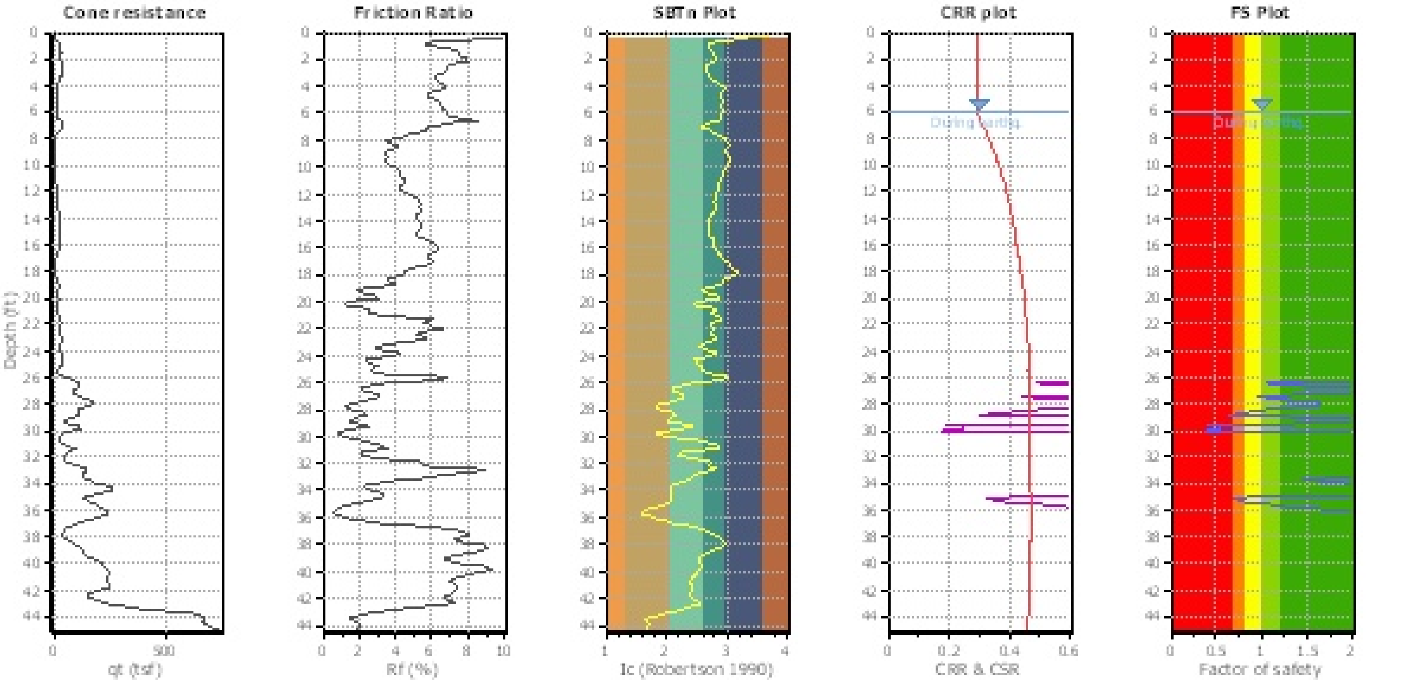
LIQUEFACTION ANALYSIS REPORT

Project title : 13081.02 - Business Center Drive Apartments **Location : Fairfield**

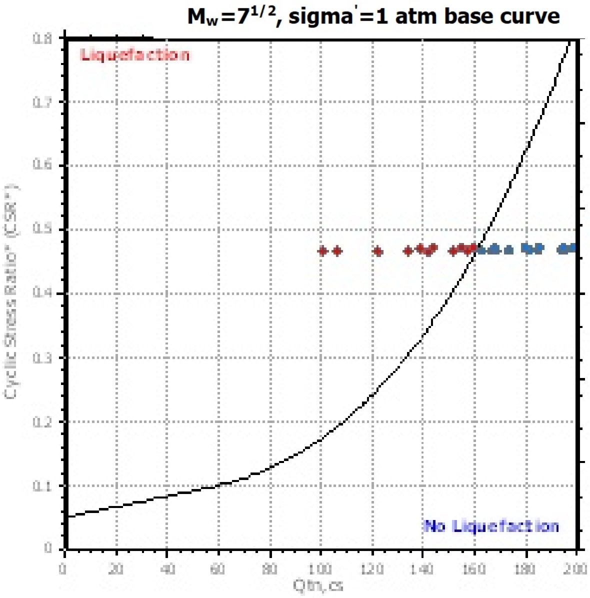
CPT file : CPT-03

Input parameters and analysis data

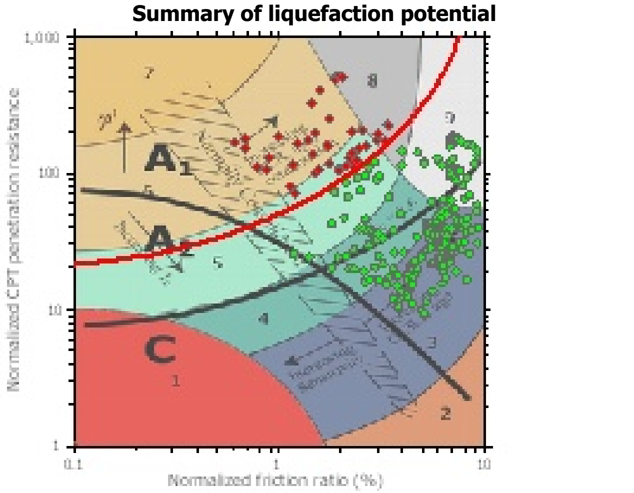
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	8.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	6.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	60.00 ft
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.15	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.60	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		



$M_w=7^{1/2}$, sigma'=1 atm base curve



Summary of liquefaction potential



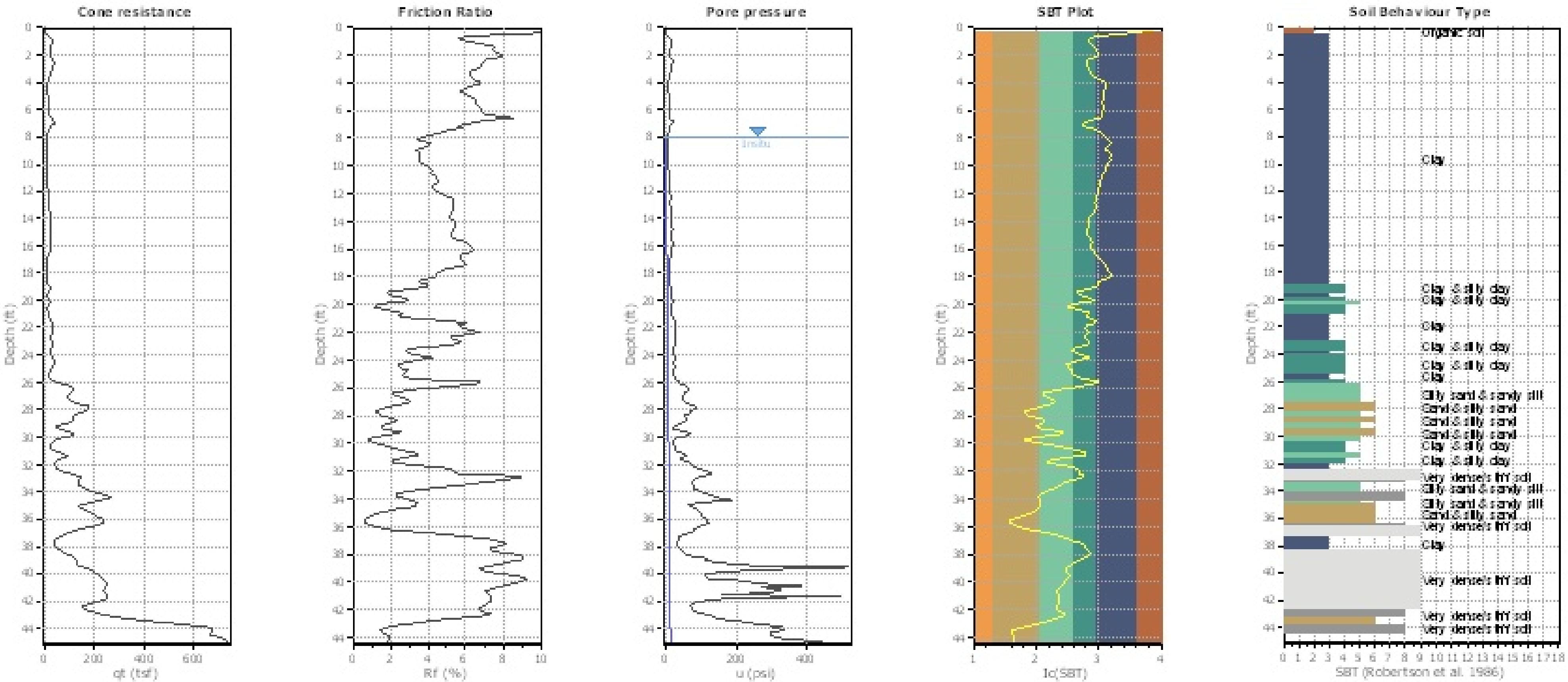
Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading

Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry

Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening

Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plo



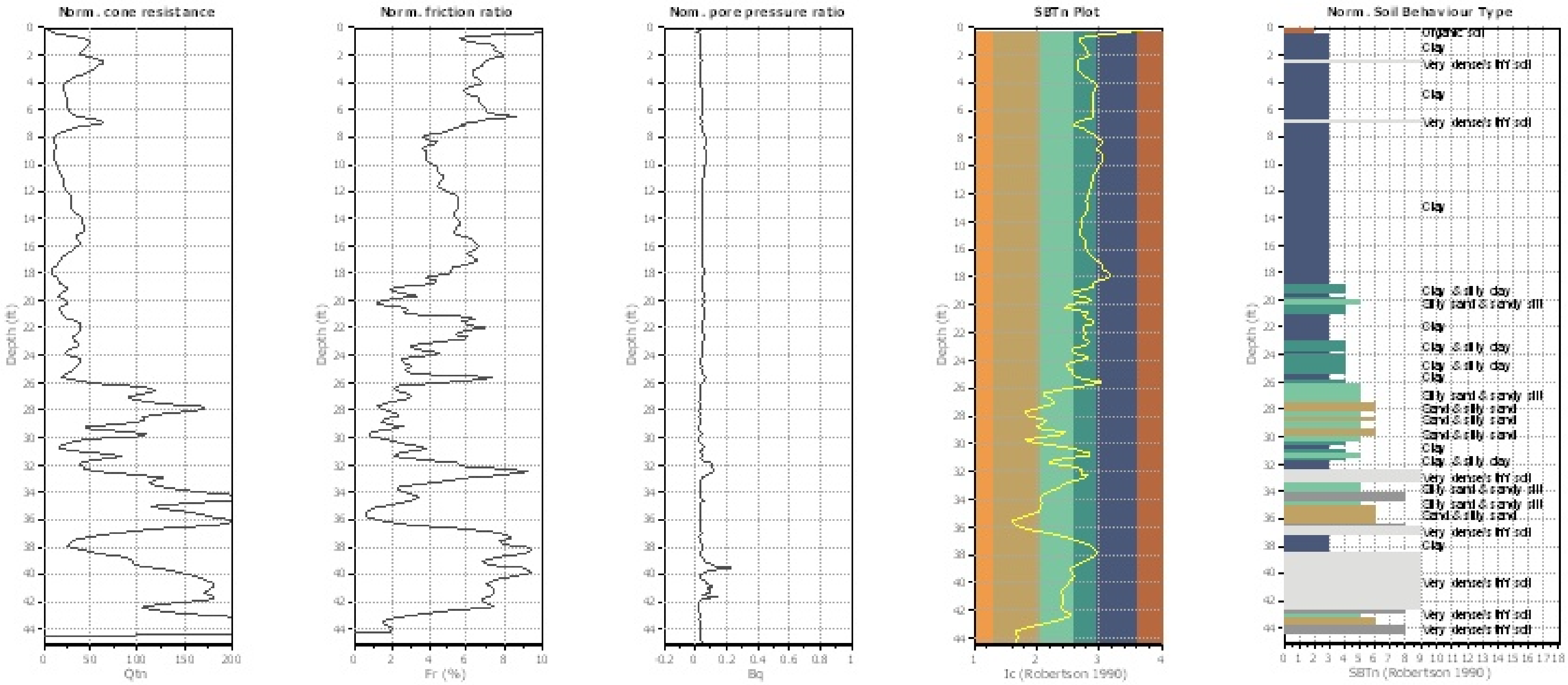
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	6.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.15	K_σ applied:	Yes
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

CPT basic interpretation plots (normaliz



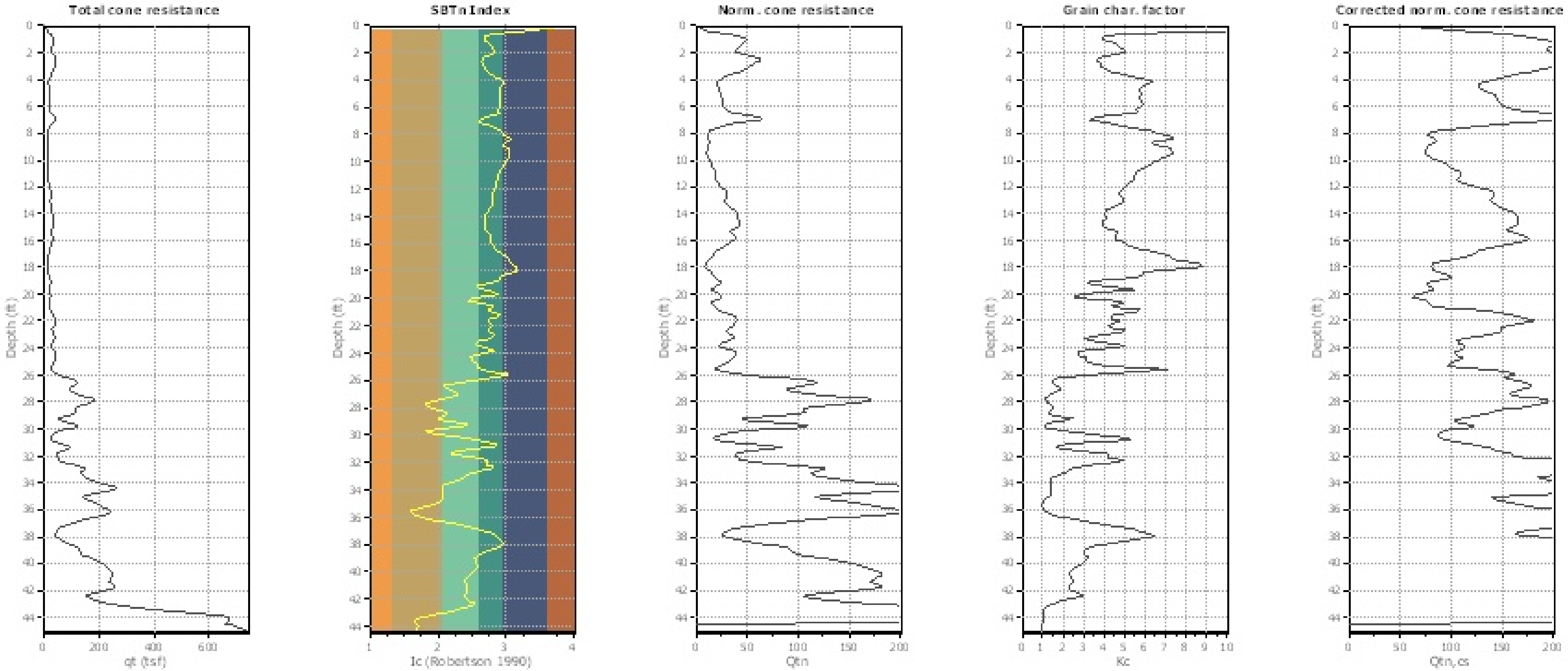
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	6.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.15	K_σ applied:	Yes
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

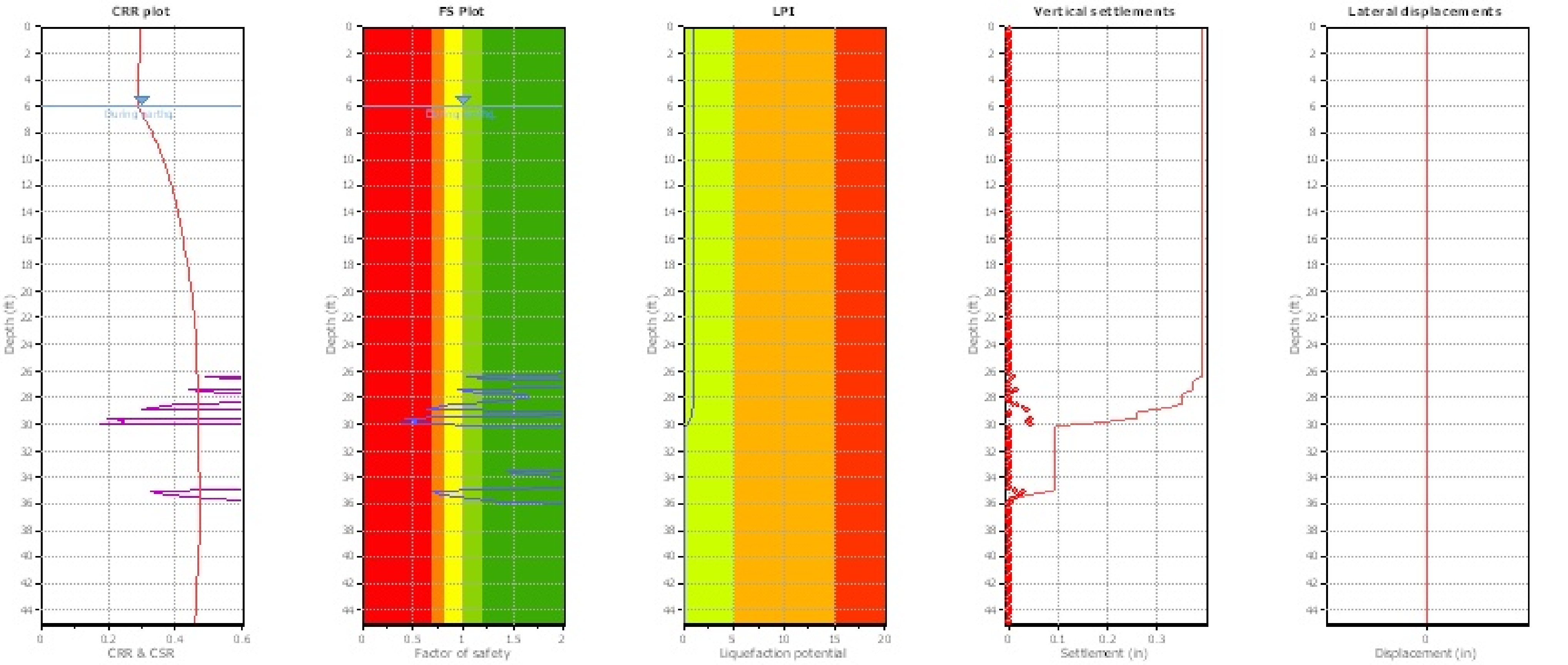
Liquefaction analysis overall plots (intermediate resu



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	6.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.15	K_σ applied:	Yes
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	6.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.15	K_σ applied:	Yes
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

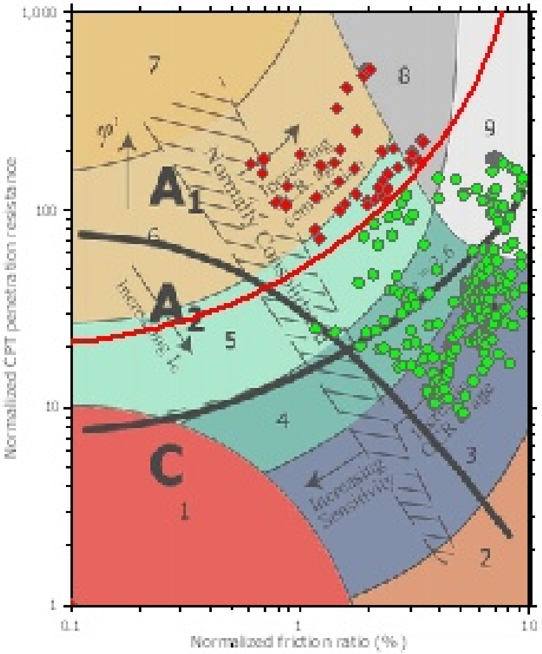
F.S. color scheme

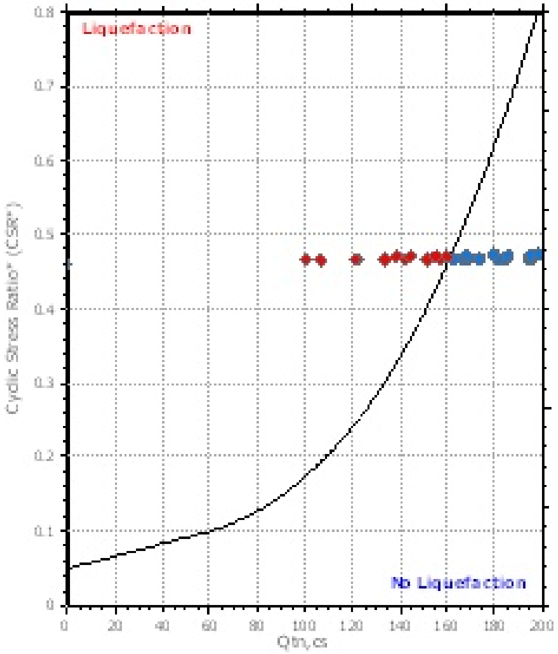
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

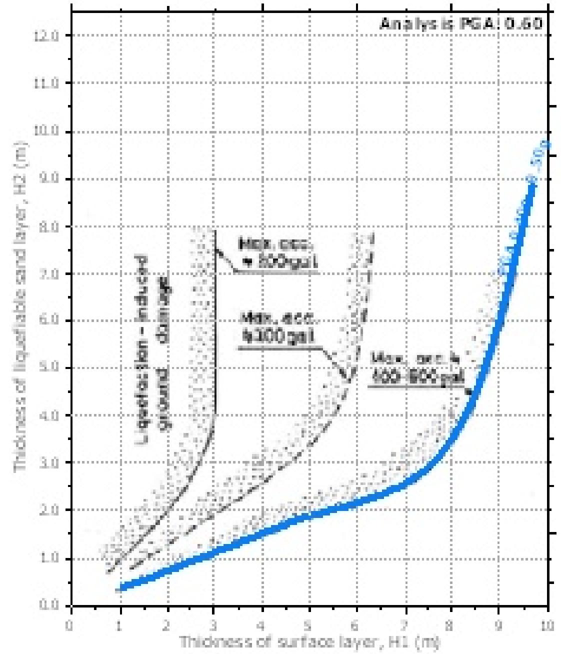
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plo



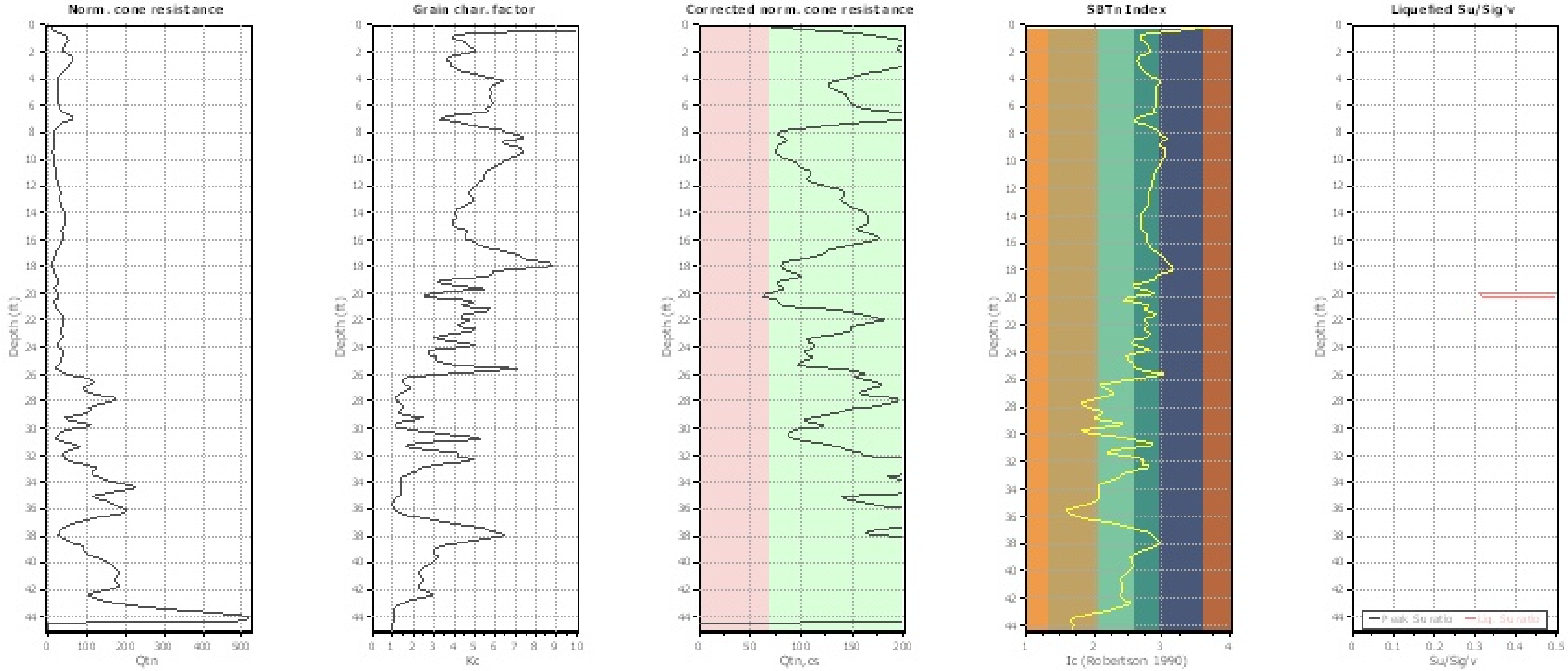




Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	6.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.15	K_σ applied:	Yes
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

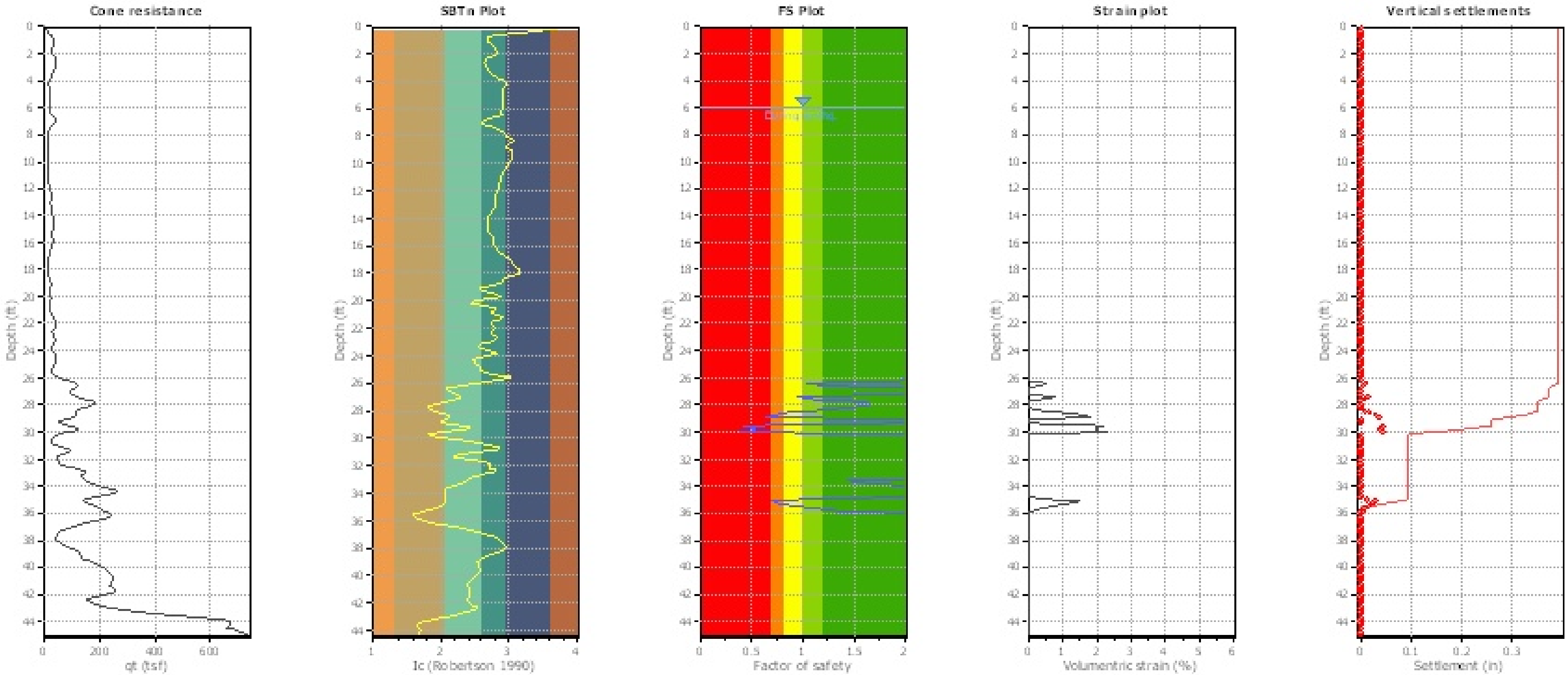
Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	6.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.15	K_σ applied:	Yes
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.60	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

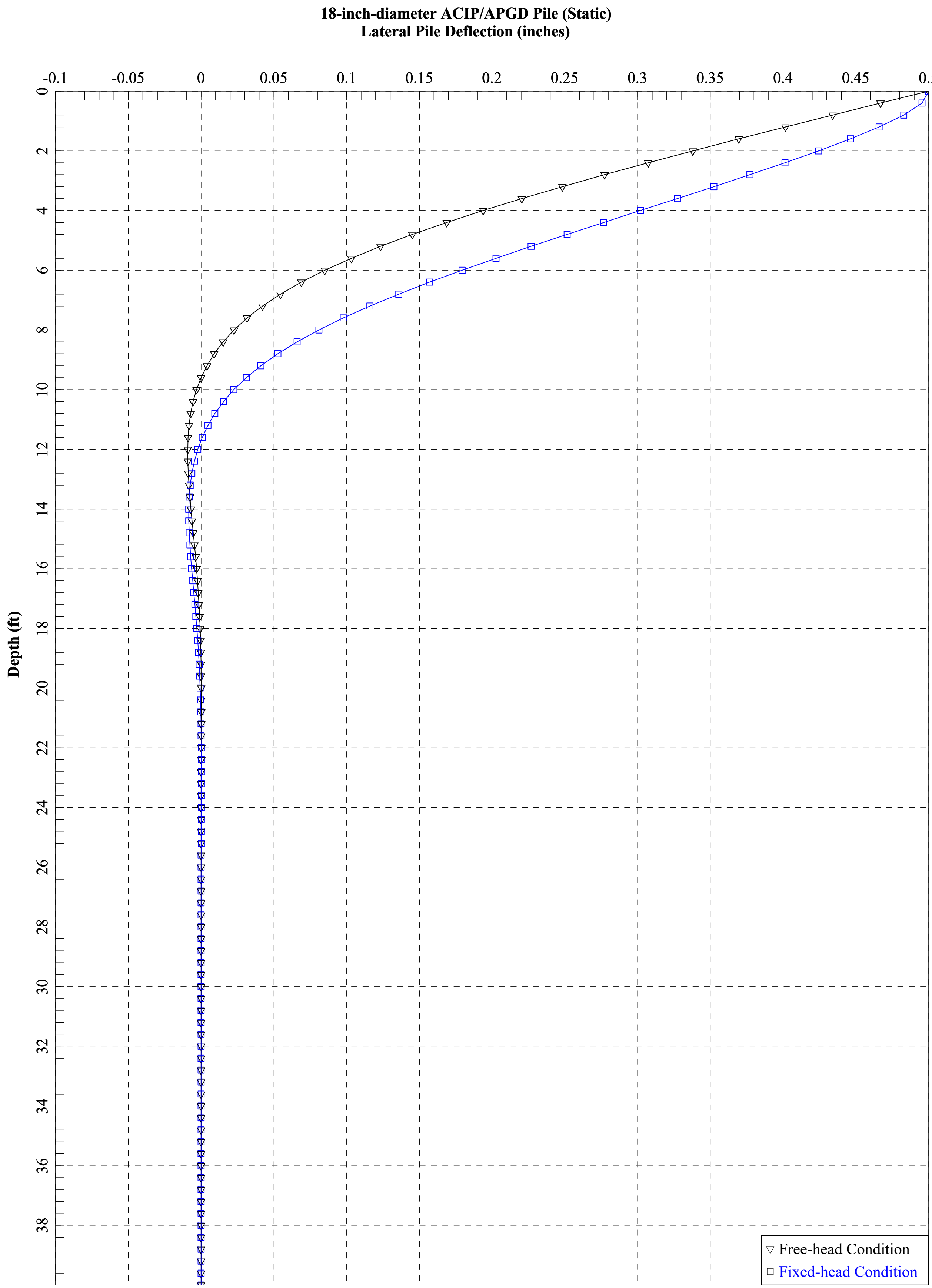
Estimation of post-earthquake settlements



Abbreviations

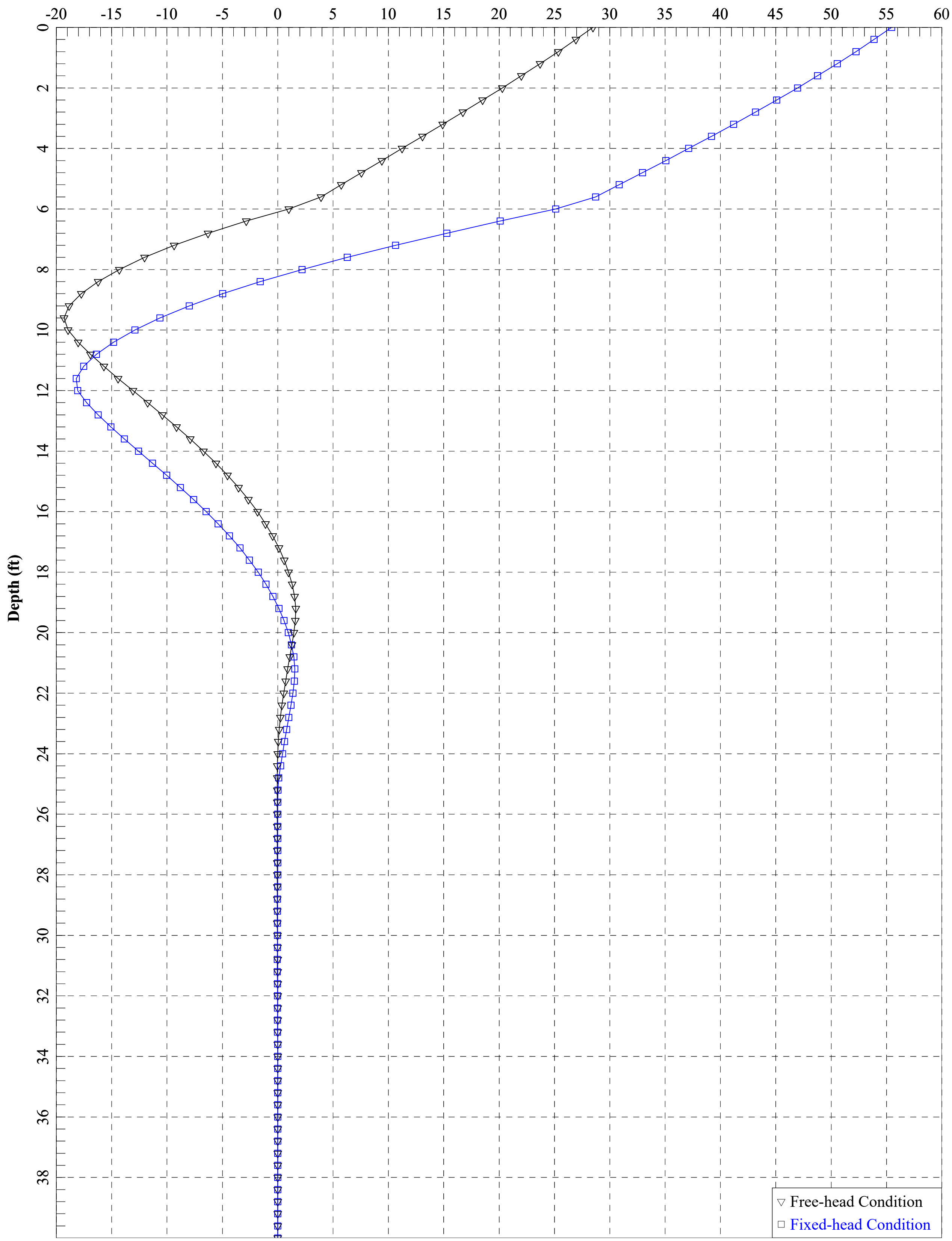
q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

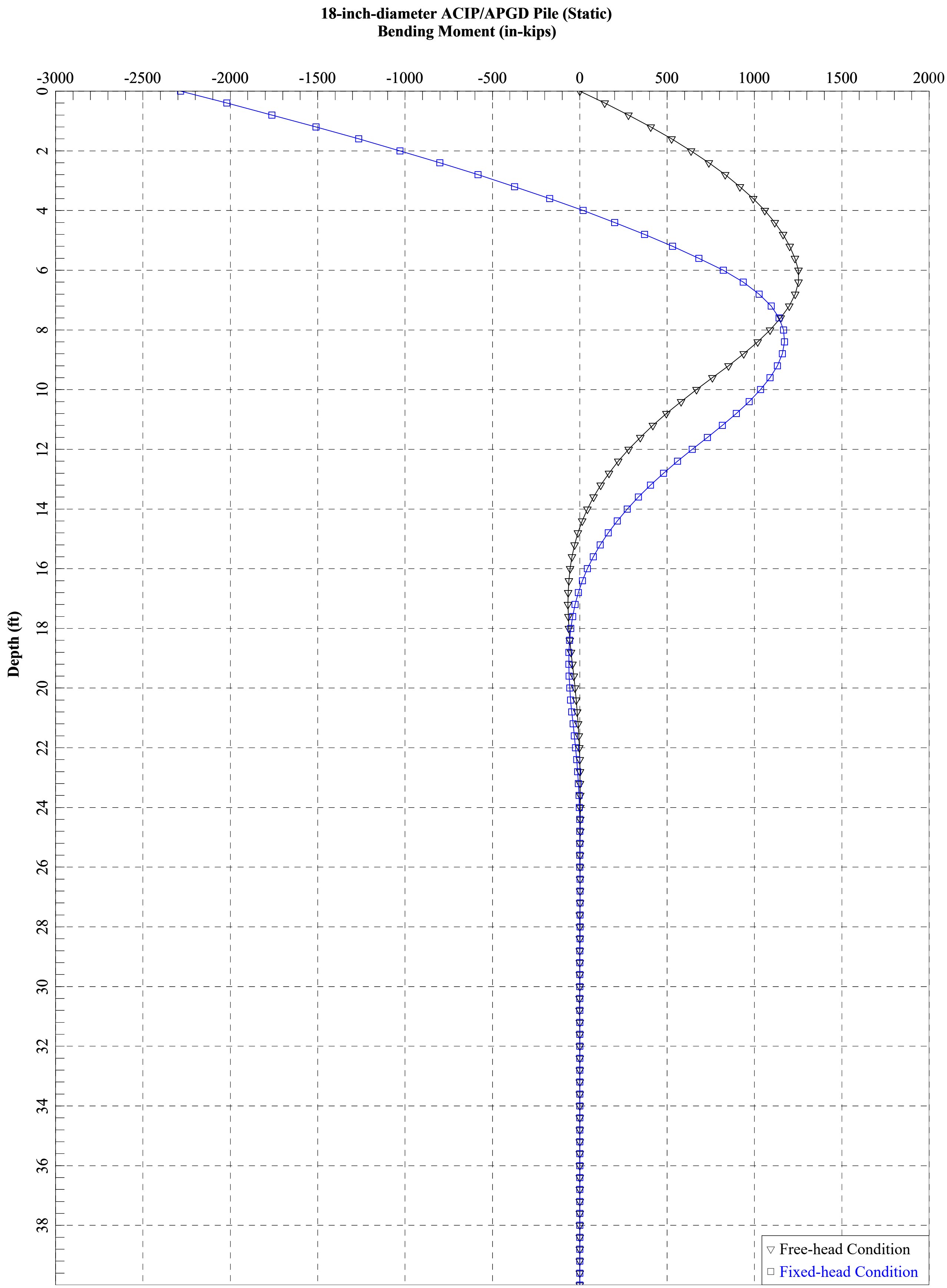
APPENDIX D
L-Pile Analysis Output Files



18-inch-diameter ACIP/APGD Pile (Static)
Lateral Pile Deflection (inches)
-0.1
-0.05
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
0.4
0.45
0.5
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
Free-head Condition
Fixed-head Condition

18-inch-diameter ACIP/APGD Pile (Static)
Shear Force (kips)




18-inch-diameter ACIP/APGD Pile (Static)
Bending Moment (in-kips)
-3000
-2500
-2000
-1500
-1000
-500
0
500
1000
1500
2000
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
Free-head Condition
Fixed-head Condition

APPENDIX E

Guide Earthwork Specifications

APPENDIX E

GUIDE EARTHWORK SPECIFICATIONS

GREEN VALLEY 3 APARTMENTS

Fairfield, California

WKA No. 13081.02

PART 1: GENERAL

1.1 SCOPE

A. General Description

This item shall include all clearing of existing surface and subsurface structures, utilities, vegetation, rubbish, rubble and associated items; preparation of surfaces to be filled, filling, spreading, compaction, observation and testing of the fill; and all subsidiary work necessary to complete the grading of the site to conform with the lines, grades and slopes as shown on the accepted Drawings.

B. Related Work Specified Elsewhere

(1) Trenching and backfilling for sanitary sewer system: Section ___.

(2) Trenching and backfilling for storm drain system: Section ___.

(3) Trenching and backfilling for underground water, natural gas, and electric supplies: Section ___.

C. Geotechnical Engineer

Where specific reference is made to "Geotechnical Engineer" this designation shall be understood to include either the Geotechnical Engineer or his or her representative.

1.2 PROTECTION

A. Adequate protection measures shall be provided to protect workers and passers-by the site. Streets and adjacent property shall be fully protected throughout the operations.

B. In accordance with generally accepted construction practices, the Contractor shall be solely and completely responsible for working conditions at the job site, including safety of all persons and property during performance of the work. This requirement shall apply continuously and shall not be limited to normal working hours.

C. Any construction review of the Contractor's performance conducted by the Geotechnical Engineer is not intended to include review of the adequacy of the Contractor's safety measures, in, on or near the construction site.

D. Adjacent streets and sidewalks shall be kept free of mud, dirt, or similar nuisances resulting from earthwork operations.

E. Measures shall be taken to protect storm drains in adjacent depressed areas such that minimum siltation occurs in the drainage system.

F. Surface drainage provisions shall be made during the period of construction in a manner to avoid creating a nuisance to adjacent areas.

G. The site and adjacent influenced areas shall be watered as required to suppress dust nuisance.

1.3 GEOTECHNICAL REPORT

A. A *Geotechnical Engineering Report* (WKA No. 13081.02, dated May 4, 2021, revised February 16, 2022) has been prepared for this site by Wallace - Kuhl & Associates, Geotechnical Engineers of West Sacramento, California [(916) 372-1434]. A copy is available for review at the office of Wallace - Kuhl & Associates.

B. The information contained in this report was obtained for design purposes only. The Contractor is responsible for any conclusions the Contractor may draw from this report; should the Contractor prefer not to assume such risk, the Contractor should employ experts to analyze available information and/or to make additional borings upon which to base conclusions drawn by the Contractor, all at no cost to the Owner.

1.4 EXISTING SITE CONDITIONS

The Contractor shall become acquainted with all site conditions. If unshown active utilities are encountered during the work, the Architect shall be promptly notified for instructions. Failure to notify will make the Contractor liable for damage to these utilities arising from Contractor's operations subsequent to the discovery of such unshown utilities.

1.5 SEASONAL LIMITS

Fill material shall not be placed, spread, or rolled during unfavorable weather conditions. When the work is interrupted by heavy rains, fill operations shall not be resumed until field tests indicate that the moisture contents of the subgrade and fill materials are satisfactory.

PART 2: PRODUCTS

2.1 MATERIALS

A. All fill shall be of approved local materials from required excavations, supplemented by imported fill, if necessary. Approved local materials are defined as local soils that do not contain significant quantities of rubble, rubbish, and vegetation, and having been tested and approved by the Geotechnical Engineer prior to use.

B. All imported fill materials shall be approved by the Geotechnical Engineer prior to being transported to the site. All fill material also shall be free of particles not exceeding three inches (3") in maximum dimension, not contain known contaminants and be within acceptable corrosion limits, with appropriate documentation provided by the contractor.

C. Imported fill materials to be used in the upper twelve to eighteen inches (12” to 18”) of soil subgrade beneath interior floor slabs and exterior flatwork, as defined in the *Geotechnical Engineering Report*, shall be compactable, well-graded, granular soils with a Plasticity Index not exceeding fifteen (15) when tested in accordance with ASTM D4318 and an Expansion Index not exceeding twenty (20) when tested in accordance with ASTM D4829.

D. Imported fill materials to be used deeper than the twelve to eighteen inches (12” to 18”) of the final soil subgrade elevation beneath interior floor slabs or exterior flatwork, as defined in the *Geotechnical Engineering Report*, within pavement areas, or to be lime-treated and used beneath interior floor-slabs, exterior flatwork, swimming pool deck slabs, or pavements can consist of locally derived clay soils provided they are compactable, possess a Plasticity Index not exceeding forty (40) and a Liquid Limit not exceeding sixty-four (64) when tested in accordance with ASTM D4318 and possess an Expansion Index not exceeding one hundred and twenty-three (123) when tested in accordance with ASTM D4829.

E. Materials to be lime-treated shall be on-site or approved imported clay soils free from significant quantities of rubble, rubbish and vegetation and shall have been tested and approved by the Geotechnical Engineer.

F. Capillary barrier material under floor slabs shall be provided to the thickness shown on the Drawings. This material shall be clean gravel or crushed rock of one-inch (1") maximum size, with less than five percent (5%) material passing a Number Four (#4) sieve.

G. Lime used for stabilization shall be high-calcium or dolomitic quicklime conforming to the definitions in ASTM Designation C977.

1) When sampled by the Geotechnical Engineer from the lime spreader or during the spreading operations, the sample of lime shall conform to the following requirements:

Lime Quality

Property	ASTM Designation	Requirements
Available calcium and magnesium oxide [minimum percent (%)]	C25 or C1301 & C1271	High calcium quicklime: CaO > 90% Dolomitic quicklime: CaO > 55% & CaO + MgO > 90%
Loss on ignition [maximum percent (%)]	C25	7% (total loss) 5% (carbon dioxide) 2% (free moisture)
Slaking Rate [degrees Celsius (°C)]	C110	30°C rise in 8 minutes

2) When dry sieved in a mechanical sieve shaker for 10 minutes ±30 seconds, a 0.5 pound (lb) test sample of quicklime shall conform to the following grading requirements:

Lime Grading

Sieve Sizes	Percentage Passing
3/8-inch	98 - 100

H. The burden of proof as to quality and suitability of alternatives shall be upon the Contractor and/or Supplier and he shall furnish test data and all information necessary, as required by the Geotechnical Engineer. Written request for alternatives, accompanied by complete data as to the quality and suitability of the material shall be made in ample time to permit testing and approval without delaying the work. The Geotechnical Engineer shall be the sole judge as to the quality and suitability of alternatives and his decision shall be final.

Documentation shall be provided to the Geotechnical Engineer no later than two weeks before the alternative material is imported to the site.

I. Lime from more than one source or of more than one type may be used on the same project but the different limes shall not be mixed.

J. The lime shall be protected from moisture until used and shall be sufficiently dry to flow freely when handled.

K Water for use in subgrade stabilization shall be clean and potable and shall be added during mixing, remixing, and compaction operations, and during the curing period to keep the cured material moist until covered.

L. Other products, such as aggregate base, asphalt concrete and related asphaltic seal coats, tack coat, etc., shall comply with the appropriate provision of the State of California (Caltrans) Standard Specifications, latest edition.

PART 3: EXECUTION

3.1 LAYOUT AND PREPARATION

Lay out all work, establish grades, locate existing underground utilities, set markers and stakes, set up and maintain barricades and protection of utilities prior to beginning actual earthwork operations.

3.2 CLEARING, STRIPPING, AND PREPARING BUILDING PAD AND PAVEMENT AREAS

A. All surface and other sub-surface items at the site, including utilities and associated backfill, vegetation, debris, and other items encountered during site work and deemed unacceptable by the Geotechnical Engineer, shall be removed and disposed of so as to leave the disturbed areas with a neat and finished appearance, free from unsightly debris. Any vegetation designated for removal shall include the rootball and all surface roots larger than one-half inch (½") in diameter. Adequate removal of debris and roots may require laborers and handpicking to clean the subgrade soils to the satisfaction of the Geotechnical Engineer's on-site representative, prior to further site preparation. All demolition debris shall be hauled off site.

B. If any wells, septic systems or tanks are encountered at the site, they shall be properly abandoned in accordance with Solano County requirements.

C. Excavations and depressions resulting from the removal of such items, as determined by the Geotechnical Engineer, shall be cleaned out to firm, undisturbed soils and backfilled with suitable materials in accordance with these specifications.

D. All structural areas (building pads, pavements, exterior flatwork, etc.) shall be stripped of vegetation and organically laden topsoil. With prior approval of our office, stripping may be used in landscaped areas, provided they are kept at least five feet (5') from buildings pads and other structural improvements, moisture conditioned and compacted.

E. Over-excavation to remove soils from within the building pads shall be performed as recommended in the *Geotechnical Engineering Report*. The extents of the required excavation shall be clearly marked on the final civil engineering or grading plans.

F. After over-excavation operations have been performed, the Geotechnical Engineer's representative shall evaluate the exposed subgrade to determine if additional over-excavation is required due to disturbed subgrade soils or if watering of the exposed subgrade is required to mitigate desiccation crack deeper than twelve inches (12") below the exposed subgrades.

G. If desiccation cracks within subgrades resulting from the required over-excavation are less than twelve inches (12") deep, the bottom of the required excavations, as well as areas to receive fill, achieved by excavation or remain at grade, shall be scarified to a depth of at least twelve inches (12"), uniformly moisture conditioned to at least two percent (2%) above the optimum moisture content, and uniformly compacted to at least ninety percent (90%) of the maximum dry density as determined by ASTM D1557 Test Method.

H. Compaction operations for all soil subgrades shall be undertaken with a heavy, self-propelled, sheepsfoot compactor capable of achieving the compaction requirements included in the Geotechnical Engineering Report.

I. When the moisture content of the fill material is less than the optimum moisture content for granular/silty soils or at least two percent (2%) above the optimum moisture content for clay soils as defined by the ASTM D1557 Compaction Test, water shall be added until the proper moisture content is achieved.

J. When the moisture content of the subgrade is too high to permit the specified compaction to be achieved, the subgrade shall be aerated by blading or other methods until the moisture content is satisfactory for compaction.

K. Site clearing and subgrade preparation operations shall extend at least five feet (5') beyond the building pads, exterior flatwork areas, pavements and any other structural areas.

L. Compaction operations shall be performed in the presence of the Geotechnical Engineer who will evaluate the performance of the materials under compactive load. Loose, soft, and saturated soils and unstable soil deposits, as determined by the Geotechnical Engineer, shall be excavated to expose a firm base and grades restored with engineered fill in accordance with these specifications.

3.3 CONSTRUCTION OF UNTREATED SUBGRADES

A. The selected soil fill material shall be placed in layers which when compacted shall not exceed six inches (6") in compacted thickness. Each layer shall be spread evenly and shall be thoroughly mixed during the spreading to promote uniformity of material in each layer.

B. When the moisture content of the fill material is less than the optimum moisture content for granular/silty soils or at least two percent (2%) above the optimum moisture content for clay soils, as defined by the ASTM D1557 Compaction Test, water shall be added until the proper moisture content is achieved.

C. When the moisture content of the fill material is too high to permit the specified degree of compaction to be achieved, the fill material shall be aerated by blading or other methods until the moisture content is satisfactory.

D. After each layer has been placed, mixed and spread evenly, it shall be thoroughly compacted to at least ninety percent (90%) as determined by the ASTM D1557 Compaction Test. Compaction shall be undertaken with equipment capable of achieving the specified density and shall be accomplished while the fill material is at the required moisture content. Each layer shall be compacted over its entire area until the desired density has been obtained.

E. Fill materials within the within the footprint of buildings that are deeper than five feet below the final soil subgrade elevation, if any, should be compacted to at least 92 percent relative compaction, at a moisture content of at least the optimum moisture content for granular soils and at least two percent above the optimum moisture content for clay soils.

F. The filling operations shall be continued until the fills have been brought to the finished slopes and grades as shown on the accepted Drawings.

3.4 LIME-STABLIZED SUBGRADE CONSTRUCITON

A. On-site or approved imported clay material, or sand/silt material blended with clay materials, to be treated shall be placed at a moisture content at least two percent (2%) over optimum moisture as defined by the ASTM D1557 Compaction Test.

B. Material to be treated shall be scarified and thoroughly broken up to the full depth and width to be stabilized. The material to be treated shall contain no rocks or solids larger than one and one-half inches (1½") in maximum dimension.

C. Mixing lime-treated material shall consist of the following:

1) Lime shall be added to the material to be treated at a rate of no less than four and a half pounds (4½ lbs.) of lime per square foot of treated soil, at a depth sufficient to produce a compacted lime-treated layer twelve inches (12”) thick, or six and three quarter pounds (6¾ lbs.) of lime per square foot of treated soil, at a depth sufficient to produce a compacted lime-treated layer eighteen inches (18”) thick.

2) Lime shall be spread by equipment that will uniformly distribute the required amount of lime for the full width of the prepared material. The rate of spread per linear foot of blanket shall not vary more than five percent (5%) from the designated rate.

3) The spread lime shall be prevented from blowing by suitable means selected by the Contractor. Quicklime shall not be used to make lime slurry. The spreading operations shall be conducted in such a manner that a hazard is not present to construction personnel or the public. All lime spread shall be thoroughly ripped in, or mixed into, the soil the same day lime spreading operations are performed.

4) The distance which lime may be spread upon the prepared material ahead of the mixing operation shall be determined by the Geotechnical Engineer.

5) No traffic other than the mixing equipment will be allowed to pass over the spread lime until after the completion of mixing.

6) Mixing equipment shall be equipped with a visual depth indicator showing mixing depth, an odometer or foot meter to indicate travel speed and a controllable water additive system for regulating water added to the mixture.

7) Mixing equipment shall be of the type that can mix the full depth of the treatment specified and leave a relatively smooth bottom of the treated section. Mixing and re-mixing, regardless of equipment used, will continue until the

material is uniformly mixed (free of streaks or pockets of lime), moisture is at approximately two percent (2%) over optimum, and the mixture complies with the following requirements:

Sieve Size	Minimum Percent Passing
1-1/2"	100
1"	95
No. 4	60

8) Non-uniformity of color reaction when the treated material, exclusive of one inch or larger clods, as tested with the standard phenolphthalein alcohol indicator, will be considered evidence of inadequate mixing.

9) Lime-treated material shall not be mixed or spread while the atmospheric temperature is below 35 degrees Fahrenheit (35°F).

10) Remixing of the treated soils shall be performed no sooner than twelve (12) hours after the initial mixing, and no later than seventy-two (72) hours after the initial mixing. The entire mixing operation shall be completed within seventy-two (72) hours of the initial spreading of lime, unless otherwise permitted by the Geotechnical Engineer.

D. Spreading and compacting of lime-treated material shall consist of the following:

1) The treated mixture shall be spread to the required width, grade, and cross-section. The maximum compacted thickness of a single layer may be determined by the Contractor provided he can demonstrate to the Geotechnical Engineer that his equipment and method of operation will provide uniform distribution of the lime and the required compacted density throughout the layer. If the Contractor is unable to achieve uniformity and density throughout the thickness selected, he shall rework the affected area using thinner lifts until a satisfactory treated subgrade meeting the distribution and density requirements is attained, as determined by the Geotechnical Engineer, at no additional cost to the Owner.

2) The finished thickness of the lime-treated material shall not vary more than one-tenth foot (0.1') from the planned thickness at any point.

3) The lime-treated soils shall be compacted to a relative compaction of not less than ninety percent (90%) for structural areas (concrete foundation slabs, exterior

flatwork, etc.) and ninety five percent (95%) for pavements as determined by the ASTM D1557 Compaction Test.

4) Initial compaction shall be performed by means of a sheepsfoot or segmented wheel roller. Final rolling shall be by means of steel-tired or pneumatic-tired rollers.

5) Areas inaccessible to rollers shall be compacted to meet the minimum compaction requirement by other means satisfactory to the Geotechnical Engineer.

6) Final compaction shall be completed within thirty-six (36) hours of initial mixing, and within four (4) hours of final mixing. The surface of the finished lime-treated material shall be the grading plane and at any point shall not vary more than eight one hundredths of a foot (0.08') foot above or below the grade established by the Civil Engineer except that when the lime-treated material is to be covered by material which is paid for by the cubic yard the surface of the finished lime-treated material shall not extend above the grade established by the Civil Engineer.

7) Before final compaction, if the treated material is above the grade tolerance specified in this section, uncompacted excess material may be removed and used in areas inaccessible to mixing equipment. After final compaction and trimming, excess material shall be removed and disposed of off-site. The trimmed and completed surface shall be rolled with steel or pneumatic-tired rollers. Minor indentations may remain in the surface of the finished material so long as no loose material remains in the indentations.

8) At the end of each day's work, a construction joint shall be made in thoroughly compacted material and with a vertical face. After a partial-width section has been completed, the longitudinal joint against which additional material is to be placed shall be trimmed approximately three inches (3") into treated material, to the neat line of the section, with a vertical edge. The material so trimmed shall be incorporated into the adjacent material to be treated.

9) An acceptable alternate to the above construction joints, if the treatment is performed with cross shaft rotary mixers, is to actually mix three inches (3") into the previous day's work to assure a good bond to the adjacent work.

3.5 FINAL SUBGRADE PREPARATION USING UNTREATED SOILS

A. Final subgrade for the apartment and clubhouse building pads and exterior flatwork areas shall be constructed in accordance with Section 3.2 and Section 3.3 of these specifications. Clay soils shall not be used in fills within the upper twelve inches (12”) of the final subgrade for all apartment and clubhouse building pads and exterior flatwork areas (except swimming pool deck slabs), unless the lime-treatment alternative included in the *Geotechnical Engineering Report* is selected. The upper twelve inches (12") of final subgrade for the apartment and clubhouse building pads and exterior flatwork areas shall consist of imported, compactable, very-low expansive (Expansion Index equal to or less than 20), granular soil, be brought to a uniform moisture content not less than the optimum moisture content, and shall be uniformly compacted to not less than ninety percent (90%) as determined by ASTM D1557 Compaction Test.

B. Final subgrade for swimming pool deck slab areas shall be constructed in accordance with Section 3.2 and Section 3.3 of these specifications. Clay soils shall not be used in fills within the upper eighteen inches (18”) of the final subgrade for swimming pool deck slab areas, unless the lime-treatment alternative included in the *Geotechnical Engineering Report* is selected. The upper eighteen inches (18") of final subgrade for all swimming pool deck slab areas shall consist of imported, compactable, very-low expansive (Expansion Index equal to or less than 20), granular soil, be brought to a uniform moisture content not less than the optimum moisture content, and shall be uniformly compacted to not less than ninety percent (90%) as determined by ASTM D1557 Compaction Test.

C. Final subgrade for pavements, including those associated with the parking structure, shall be constructed in accordance with Section 3.2 and Section 3.3 of these specifications. The upper twelve inches (12”) of untreated final pavement subgrades shall be brought to a uniform moisture content of at least two percent (2%) above the optimum moisture content, and shall be uniformly compacted to not less than ninety-five percent (95%) as determined by ASTM D1557 Compaction Test, regardless of whether final subgrade elevations are attained by filling, excavation or are left at existing grades.

3.6 FINAL SUBGRADE PREPARATION USING TREATED SOILS

A. Final subgrade for apartment and clubhouse building pads and exterior flatwork areas using treated soils shall be constructed in accordance with Section 3.2 and

Section 3.4 of these specifications. If the lime-treatment alternative is selected for finals subgrade of building pad and exterior flatwork areas (except swimming pool deck slabs), the upper twelve inches (12”) of treated final subgrades shall be brought to a uniform moisture content of at least two percent (2%) above the optimum moisture content, and shall be uniformly compacted to not less than ninety percent (90%) as determined by ASTM D1557 Compaction Test, regardless of whether final subgrade elevations are attained by filling, excavation or are left at existing grades.

B. Final subgrade for swimming pool deck areas using treated soils shall be constructed in accordance with Section 3.2 and Section 3.4 of these specifications. If the lime-treatment alternative is selected for the swimming pool deck slab areas, the upper eighteen inches (18”) of treated final subgrades shall be brought to a uniform moisture content of at least two percent (2%) above the optimum moisture content, and shall be uniformly compacted to not less than ninety percent (90%) as determined by ASTM D1557 Compaction Test, regardless of whether final subgrade elevations are attained by filling, excavation or are left at existing grades.

C. Final subgrade for pavements, including those associated with the parking structure, using treated soils shall be constructed in accordance with Section 3.2 and Section 3.4 of these specifications. If the lime-treatment alternative is selected for pavement subgrades, the upper twelve inches (12”) of treated pavement subgrades shall be brought to a uniform moisture content of at least two percent (2%) above the optimum moisture content, and shall be uniformly compacted to not less than ninety-five percent (95%) as determined by ASTM D1557 Compaction Test, regardless of whether final subgrade elevations are attained by filling, excavation or are left at existing grades.

3.7 TESTING AND OBSERVATION

A. Grading operations, including lime-treatment of subgrades, shall be observed by the Geotechnical Engineer, serving as the representative of the Owner.

B. Field density tests shall be made by the Geotechnical Engineer after compaction of each layer of fill. Additional layers of fill shall not be spread until the field density tests indicate that the minimum specified density has been obtained.

C. Earthwork shall not be performed without the notification or approval of the Geotechnical Engineer. The Contractor shall notify the Geotechnical Engineer at least two (2) working days prior to commencement of any aspect of the site earthwork.

D. If the Contractor should fail to meet the technical or design requirements embodied in this document and on the applicable plans, the necessary readjustments shall be made by the Contractor until all work is deemed satisfactory, as determined by the Geotechnical Engineer and the Architect/Engineer. No deviation from the specifications shall be made except upon written approval of the Geotechnical Engineer or Architect/Engineer.

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APPENDIX F
Guide Specifications for Auger Cast-in-Place (ACIP) Piles

APPENDIX F

GUIDE SPECIFICATIONS FOR AUGER CAST-IN-PLACE (ACIP) PILES

GREEN VALLEY 3 APARTMENTS

Fairfield, California

WKA No. 13081.02

PART 1: GENERAL

1.1 SUMMARY

A. This Section includes construction of compression and tension auger cast piles, where shown on contract drawings and specified herein.

B. The Contractor shall furnish all labor, materials, tools, and equipment necessary for designing, furnishing, installing, inspecting and testing augered cast-in-place piles, and shall remove and dispose spoils generated by pile construction.

1.2 WORK NOT INCLUDED UNDER THIS SECTION

A. Concrete pile caps: Section _______.

B. Excavations: Section ______.

C. Shoring and bracing of earth banks: Section ______.

D. Dewatering: Section ______.

1.3 REFERENCE STANDARDS

A. Requirements, abbreviations and acronyms for reference standards are defined in Section _____.

B. American Concrete Institute (ACI)

1. ACI 305 - Hot Weather Concreting.
2. ACI 306 - Cold Weather Concreting.
3. ACI 315 - Details and Detailing of Concrete Reinforcement.

C. American Society for Testing and Materials (ASTM) latest editions

1. ASTM A 615 - Deformed and Plain Billet-Steel Bars for Concrete Reinforcement.
2. ASTM C 33 - Concrete Aggregates.
3. ASTM C 31 - Standard Practice for Making and Curing Concrete Test Specimens in the Field

4. ASTM C 109 - Test Method for Compressive Strength of Hydraulic Cement Mortars.
5. ASTM C 150 - Portland Cement.
6. ASTM C 618 - Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete.
7. ASTM C 939 - Test Method for Flow of Grout for Preplaced - Aggregate Concrete (Flow Cone Method)
8. ASTM C 942 - Test Method for Compressive Strength of Grouts for Preplaced-Aggregate Concrete in the Laboratory.
9. ASTM D 1143 - Test Method for Piles Under Static Axial Compressive Load.
10. ASTM D 3689 - Test Method for Individual Piles Under Static Axial Tensile Load.
11. ASTM D 3966 - Test Method for Piles Under Lateral Loads.

1.4 PROTECTION

A. Adequate protection measures shall be provided to protect workers and passers-by at the site. Streets and adjacent property shall be fully protected throughout the operations.

B. In accordance with generally accepted construction practices, the Contractor shall be solely and completely responsible for working conditions at the job site, including safety of all persons and property during performance of the work. This requirement shall apply continuously and shall not be limited to normal working hours.

C. Any construction review of the Contractor's performance conducted by the Geotechnical Engineer is not intended to include review of the adequacy of the Contractor's safety measures, in, on or near the construction site.

D. Adjacent streets and sidewalks shall be kept free of mud, dirt or similar nuisances resulting from earthwork operations.

E. Surface drainage provisions shall be made during the period of construction in a manner to avoid creating a nuisance to adjacent areas.

F. The site and adjacent influenced areas shall be watered as required to suppress dust nuisance.

1.5 EXISTING SITE CONDITIONS

Piling Contractor shall inspect the site and related conditions prior to commencing their portion of the work. If unshown active utilities are encountered during the work, the Architect shall be promptly notified for instructions. Failure to notify will make the Contractor liable for damage to these utilities arising from Contractor's operations subsequent to the discovery of such unshown utilities.

1.6 GEOTECHNICAL ENGINEERING REPORT

A. A *Geotechnical Engineering Report* (WKA No. 13081.02, dated May 4, 2021, revised February 16, 2022), has been prepared by Wallace - Kuhl & Associates, Geotechnical Engineers of West Sacramento, California; telephone (916) 372-1434; facsimile (916) 372-2565. That report is available for review at the office of Wallace - Kuhl & Associates.

B. The Piling Contractor shall submit in writing to the Architect and/or Structural Engineer, all applicable information as listed in Subsection 1.7 - Submittals for review and approval, in addition to the above experience record.

C. The Owner does not guarantee that the information contained in the *Geotechnical Engineering Report* is correct nor that the conditions revealed at the actual exploration locations will be continuous over the entire site. This report was prepared for purposes of design only. Making the report available to contractors shall not be construed in any way as a waiver of this position. The Piling Contractor shall be responsible for any conclusions the Contractor may draw from this report. Should the Contractor prefer not to assume such risk, the Contractor is under obligation to employ their own experts to analyze available information and/or to make their own tests upon which to base their conclusions.

1.7 SUBMITTALS

Submit the following according to Conditions of the Construction Contract and Division 1 Specifications, for Owner's approval.

A. Shop Drawings: Shall clearly indicate but not be limited to:

1. Description of the pile drilling and grouting equipment and procedures to be utilized in installations.

2. Proposed pile grout design mix and description of materials to be used in sufficient detail to indicate their compliance with the specifications and either;
   a. Laboratory tests of trial mixes made with the proposed mix, or
   b. Laboratory tests of the proposed mix used on previous projects.
3. A pile layout plan referenced to the structural plans including a numbering system capable of identifying each individual pile and indicating pile cutoff elevations.
4. A dimensioned sketch of the pile load test arrangements, including sizes of primary members, data on testing and measuring equipment including required jack and gauge calibrations, load cell and professional engineer seal certifying the adequacy of the reaction frames.
5. Fabrication and installation schedule covering test pile installation, pile testing, and production pile installation, with excavation schedule for pile cap and finished subgrades by area.
6. Qualifications of pile installation construction personnel, supervisor, and technician.

B. Records
1. The Contractor shall submit a pile design report indicating construction methods and materials which will be utilized to install piles of the specified compression and tension capacity, meeting the criteria of this specification and the Contract Drawings. The report shall be prepared and sealed by a Professional Engineer licensed in the state of California.
2. The Contractor shall provide a Technician for each pile rig responsible for observing the auger construction, grout batching, and grouting operations and preparing installation records. The Contractor's inspector shall submit an installation record for each pile not later than two (2) days after installation is completed. The report shall include but not be limited to:
   a. Project name and number
   b. Name of contractor
   c. Pile number
   d. Pile location, date and time of installation
   e. Design pile capacity, compression or tension
   f. Pile diameter

g. Tip elevation

h. Cut off elevation

i. Elevation of butt

j. Drilling elevation

k. Rate of advancement of auger and rotation speed

l. Quantity of grout placed as compared to the theoretical volume for each pile, in five-foot (5') depth increments, and total for pile

m. Grout pressures

n. Pile reinforcing steel

o. Grout flow cone test report

p. Any unusual occurrences observed during pile installation, and pile deviation from vertical

3. The grout quantity shall be determined by recording grout pump displacement or by other acceptable means; the pile installation record shall reveal the observed measure and quantity.

4. Load test reports shall be in accordance with the applicable ASTM Standards.

5. Grout compression test reports.

C. Hazardous Materials Notification: In the event no alternative product or material is available that does not contain asbestos, polychlorinated biphenyls (PCBs) or other hazardous materials as determined by the Owners' Authorized Representative, a "Material Safety Data Sheet" (MSDS) equivalent to OSHA Form 20 shall be submitted for that proposed product or material prior to installation.

D. Asbestos and PCB Certification: After completion of installation, but prior to Substantial Completion, Contractor shall certify in writing that products and materials installed, and processes used, do not contain asbestos or polychlorinated biphenyls (PCB), using format in Section ____/Closeout Procedures.

1.8 <u>DELIVERY, HANDLING, STORAGE</u>

Comply with General Conditions and Section ____/Product Requirements.

1.9 WARRANTY

Comply with General Conditions and Section ____/Product Requirements.

PART 2: PRODUCTS

2.1 QUALITY ASSURANCE

A. The work of this section shall be performed by a company specialized in auger cast pile work with a minimum of five (5) years of documented successful experience and shall be performed by skilled workers thoroughly experienced in the necessary crafts. Contractor shall submit evidence of successful installation of augered cast-in-place piles under similar job and subsurface conditions, including a job supervisor who shall have a minimum of three (3) years of method specific experience.

B. Work shall comply with all Municipal, State and Federal regulations regarding safety, including the requirements of the California Division of Occupational Safety and Health Administration (Cal/OSHA).

2.2 MATERIALS

A. Portland Cement: conforming to ASTM C 150.

B. Mineral Admixture: Mineral admixture, if used, shall be fly ash or natural pozzolan which possesses the property of combining with the lime liberated during the process of hydration of Portland cement to form compounds containing cementitious properties, conforming to ASTM C 618, Class C or Class F.

C. Fluidifier conforming to ASTM C 937, except that expansion shall not exceed 4%.

D. Water: Potable, fresh, clean and free of sewage, oil, acid, alkali, salts or organic matter.

E. Fine Aggregate: Conforming to ASTM C 33.

F. Grout Mixes:

1. The grout shall consist of Portland cement, sand and water, and may also contain a mineral admixture and approved fluidifier.

a. The components shall be proportioned and mixed to produce a grout capable of maintaining the solids in suspension, which may be pumped without difficulty, and which will penetrate and fill open voids in the adjacent soils.

b. These materials shall be proportioned to produce a hardened grout which will achieve the design strength within twenty-eight (28) days.

c. The design grout strength at twenty-eight (28) days for this project shall be a minimum four thousand pounds per square inch (4000 psi, are fed to the mixer.

a. Time of mixing shall be not less than one minute at the site.

b. If agitated continuously, the grout may be held in the mixer or agitator for a period not exceeding two and one-half (2½) hours at grout temperatures below seventy degrees Fahrenheit (70°F) and for a period not exceeding one hundred degrees Fahrenheit (100°F).

c. Grout shall not be placed when its temperature exceeds one hundred degrees Fahrenheit (100°F).

2. Protect grout from physical damage or reduced strength, which could be caused by frost, freezing actions or low temperatures or from damage during high temperatures in accordance with ACI 305/306.

3. The grout shall be tested by making a minimum of six, two-inch (2") diameter by four-inch (4") tall cylinders for each day during which piles are placed.

   a. At minimum, a set of six (6) cylinders shall consist of one (1) cylinders tested at seven (7) days, and three (3) cylinders tested at twenty-eight (28) days. Two (2) cylinders shall be held in reserve.

   b. Test cylinders shall be cured and tested in accordance with ASTM C 109.

   c. Cylinder specimens shall be cast and cured in accordance with ASTM C 31.

   d. Cylinder specimens may be restrained from expansion as described in ASTM C 942.

4. Test the flow of grout for each pile and batch of grout. Maintain grout fluidity between fifteen (15) and twenty-five (25) seconds through a three-quarters-inch (¾") diameter grout cone.

G. Steel Reinforcing:

1. Minimum reinforcing steel assemblies are shown on the Contract Drawings. Assemblies shall be detailed and fabricated in accordance with the manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315).
2. Reinforcing shall conform to the requirements of ASTM A 615, Grade 60.
3. [All reinforcing bar shall be epoxy coated as recommended by a Corrosion Engineer, including bars installed for contractor convenience. Wire ties do not require epoxy coating.] *Use this language if a Corrosion Engineer is consulted and has determined corrosion protection is required.*
4. Contractor shall provide labor, materials, and method for coating cut ends and repairing holidays in epoxy coating.
5. Acceptable materials and methods shall be provided to facilitate proper centering of all steel reinforcing installed.
6. Bars may be bent in place, provided epoxy coating at all bends is inspected, flaked coating is removed by wire brush, and holidays in coating are repaired.
7. A corrugated metal pipe sleeve shall be provided for each pile equal to the diameter of the auger, to define the pile butt and permit cut-off to specified elevations.

2.3 <u>EQUIPMENT</u>

A. Augering Equipment:

1. The auger flighting shall be continuous from the auger head to the top of auger without gaps or other breaks.
2. The auger flighting shall be uniform in diameter throughout its length and shall be the diameter specified for the piles less a maximum of three percent (3%). The hole through which the grout is pumped during the placement of the pile shall be located at the bottom of the auger head below the bar containing the cutting teeth.
3. Augers over forty feet (40') in length shall contain a middle support guide.
4. The piling leads shall be prevented from rotating by a stabilizing arm or by firmly placing the bottom of the leads into the ground or by some other acceptable means.

5. Leads shall be marked at one-foot (1') intervals to facilitate measurement of auger penetration.
6. Auger hoisting equipment shall be provided that will enable the auger to be rotated while being withdrawn.

B. Mixing and Pumping Equipment:

1. Only approved pumping and mixing equipment shall be used in the preparation and handling of the grout.
   a. Provide a screen to remove over-size particles at the pump inlet.
   b. All oil or other rust inhibitor shall be removed from mixing drums and grout pumps before each use.
   c. All materials shall be such as to produce a homogeneous grout of the desired consistency and strength.
2. The grout pump shall be a positive displacement pump capable of developing displacement pressures at the pump of three hundred fifty pounds per square inch (350 psi) or higher.
   a. The grout pump shall be provided with a pressure gauge in clear view of the equipment operator.
   b. The grout pump shall be calibrated at the beginning of the work and periodically during the work to determine the volume of grout pumped per stroke, under operating pressure.
   c. A positive method for automatic counting of grout pump strokes shall be provided. Such methods may include digital or mechanical stroke counters or other acceptable methods.
   d. A second pressure gauge, if required, shall be provided close to the auger rig where it can be readily observed by the inspector, if required.

PART 3: EXECUTION

3.1 EXAMINATION

A. The Contractor is responsible for supporting pile drilling equipment and concrete grout batching and delivery equipment. Equipment shall be supported on timber mats or gravel fill work platforms, if necessary for safety and stability, and to prevent damage.

B. The Contractor shall examine the areas and evaluate conditions under which piles are to be installed and shall include measures for the proper and timely completion of the work in the construction methods and pile design.

3.2 AUGER CAST PILE SYSTEM DESCRIPTION

A. Augered Pressure Grouted Piles

1. Pressure grouted piles shall be made by drilling a continuous-flight, hollow-shaft auger into the ground to the design pile depth, or until refusal criteria is satisfied. The volume of soil extracted shall not be greater than the volume of the steel auger stem inserted.
2. Grout shall be injected through the auger shaft as the auger is being withdrawn. First develop a five-foot (5') plug at the bottom of the auger flights, then inject sufficient grout volume to fill the augered hole one hundred fifteen percent (115 %) of the theoretical volume or more. Grout volumes shall be logged by depth during withdrawal.
3. Post-grouting through a special grout tube for capacity increase is permitted, given these methods are used in the test piles, and consistently throughout the entire work for this project. Post-grouting may be used for compression and tension capacity. Post-grout pressures must be sufficient to open grout portals and cause fracture and flow. Grout volumes and pressures shall be recorded and used as a measure to demonstrate pile compliance with the design and pile load test criteria.

B. Augered Pressure Grouted Displacement Piles

1. Augered Pressure Grouted Displacement piles shall be made by rotating a specialized auger capable of displacing soil surrounding the auger, with minimal soils returned to the ground surface to reach the design pile depth, or until specified refusal criteria is satisfied.
2. Grout shall be injected through the auger shaft as the auger is being withdrawn in such a way as to exert a positive upward grout pressure on the auger, as well as a positive lateral pressure on the soil surrounding the pile.

C. Alternatives

1. Alternative pile types which meet the compression and tension pile criteria given on the drawings may be substituted for augered pressure grouted pile systems described in this Section.
2. Alternative pile installation systems must be capable of achieving the specified ultimate compression, tension, and lateral capacities.

3.3 <u>PILE DESIGN</u>

A. The ultimate capacity of eighteen-inch (18”) diameter compression piles shall be greater than three hundred (300) kips in axial compression and greater than one hundred fifty (150) kips in axial tension. Tension and compression piles shall achieve an ultimate lateral capacity of twenty-five (25) kips for eighteen inch (18”) diameter piles in “free-head” pile conditions and fifty (50) kips for eighteen inch (18”) diameter piles in “fixed-head” pile conditions. The allowable design capacities of all piles shall be determined by dividing the ultimate capacity by the appropriate factor of safety as provided in the *Geotechnical Engineering Report*. Load Testing performed under Part 3.4 of this section shall confirm the ultimate capacity of the piles.

B. Pile design shall be performed by the Contractor and demonstrated by load test before installation of production piles. All piles shall meet the criteria specified on the Contract Drawings.

C. The design shall be described in a pile design report. This report shall indicate variances, if any, from the reinforcing steel specified or the requirements of this section and shall demonstrate that the design meets or exceeds the specified performance in tension, compression, and bending. The Contractor shall submit design calculations for the proposed piles demonstrating compression and tensile capacity.

3.4 <u>LOAD TESTING</u>

A. Pre-construction Pile Static Load Tests:

1. Install and test one (1) compression pile, one (1) tension pile, and one (1) lateral load test pile, at the locations shown on the plans or approved

alternate location to verify the construction methods and pile capacity. Test piles and reaction piles shall be installed outside of pile cap locations.

2. The Contractor shall provide complete testing materials and equipment as required, install test and reaction piles and perform the load tests only in the presence of the Owner.
3. The pile test reaction frame shall be capable of safely sustaining at least three hundred (300) kips in axial compression and one hundred fifty (150) kips in axial tension (uplift) for eighteen-inch (18”) diameter piles.
4. Preconstruction Pile Static Load tests shall be performed using ASTM’s Quick Test Methods.
5. One successful compression pile load test shall be performed in accordance with ASTM D 1143.
6. One successful tension pile load test shall be performed in accordance with ASTM D 3689.
7. One lateral pile load test to twenty-five (25) kips ultimate load shall be performed in accordance with ASTM D 3966, assuming “free-head” pile conditions.

3.5 INSTALLATION

A. Tolerance
   1. Piles shall be located where shown on drawings or where otherwise directed by the Engineer.
      a. Pile centers shall be located to an accuracy of three inches (±3").
      b. Vertical piles shall be plumb within two percent (2%).
      c. Battered piles shall be installed to within four percent (4%) of the specified batter as determined by the angle from horizontal.

B. Adjacent Piles
   1. Adjacent piles within ten feet (10'), center-to-center, shall not be installed within twenty-four (24) hours of each other.
   2. Within pile caps, piles adjacent within four (4) pile diameters center-to-center, shall not be installed within twenty-four (24) hours of each other.

C. Installation Procedure

1. The length and drilling criteria of production piles will be as defined in the Contractor's design and as demonstrated by the successful pile load tests. Advance and rotate the auger at a continuous rate that prevents removal of excess soil.
2. Stop advancement after reaching the required depth or refusal criteria.
3. The hole in the bottom of the auger shall be closed with a suitable plug while advancing into the ground. The plug shall be removed by grout pressure or mechanically with the reinforcing bar.
4. At the start of pumping grout, raise the auger from six inches (6") to twelve inches (12") and after the grout pressure builds up sufficiently, re-drill the auger to the previously established tip elevation.
5. Maintain a head of at least fifteen feet (15') of grout on the auger flighting above the injection point during auger withdrawal.
    a. Positive rotation of the auger shall be maintained at least until placement of the grout.
    b. Rate of grout injection and rate of auger withdrawal from the soil shall be coordinated so as to maintain at all times the minimum grout head.
    c. The total volume of grout shall be at least one hundred fifteen percent (115%) of the theoretical volume for each pile.
    d. After grout is flowing at the ground surface from the auger flighting, the rate of grout injection and auger withdrawal shall be coordinated so that there is a constant grout flow at the surface.
    e. If pumping grout is interrupted for any reason, the contractor shall reinsert the auger by drilling at least five feet (5') below the depth of the auger where the interruption occurred, and re-grout while withdrawing the auger from that depth.
6. If less than one hundred fifteen percent (115%) of the theoretical volume of grout is placed in any five foot (5') increment (until the grout head on the auger flighting reaches the ground surface), the pile increment shall be reinstalled by advancing the auger ten feet (10') or to the bottom of the pile if that is less, followed by controlled removal and grout injection.
7. Spoil material that accumulates around the auger during injection of the grout shall be promptly cleared away.

8. A steel corrugated metal pipe (CMP) sleeve shall be placed at the top of each pile to a depth of one and one half feet (1½') below the pile cutoff elevation.

D. Obstructions and Damaged Piles

1. If non-augerable material is encountered above the desired tip elevation, the pile shall be completed to the depth of the non-augerable material in accordance with these Specifications. Such short piles shall be included for payment, if completed and included within the foundation. If required by the Engineer, additional adjacent piles shall be placed. Additional piles shall also be included in the total number of piles for payment.
2. Damaged piles, and piles installed outside the required installation tolerances, will not be accepted.
3. Cut off and abandon rejected piles after installation and replace with new piles. Cutoff shall be at a sufficient depth to avoid transfer of load from the structure to the abandoned pile.
4. Piles located within ten feet (10') of existing structures, if any, shall be installed in one continuous operation. Re-stroking piles during construction due to auger obstructions or difficulty in installation of reinforcement cages will <u>not</u> be allowed. The structural engineer shall be consulted in the event that replacement piles are required.

E. Cutting-Off

1. Adjust the tops of pile to the cut-off elevations where piles are constructed from a work platform above final subgrade, by removing fresh grout from the top of the pile after the CMP sleeve is in place.
2. Cut off hardened grout and the CMP shell down to final cutoff point after initial set has occurred for all piles in a single cap, or within fifteen feet (15') of any pile in a spaced pattern.

F. Disposal

1. The Contractor shall remove and dispose all spoils and grout off site with prior approval of the regulatory agencies.
2. The Contractor shall determine if any excavated material is contaminated, and if any contaminated material is encountered it shall be disposed of in a method acceptable to all governmental authorities having jurisdiction.

PART 4: MEASUREMENT AND PAYMENT

4.1 MEASUREMENT

A. Each compression pile and each tension pile successfully installed in accordance with the Contractor’s design and using the methods and practices of the approved test piles, cut off at the proper elevation, including steel reinforcing, and all records and grout testing specified, shall be considered a single unit price item. Pile design, materials testing, and the Contractor’s inspection are considered incidental to construction and shall not be separately measured for payment. Damaged piles and piles installed outside the required installation tolerances will not be measured for payment. Short piles caused by obstructions and meeting the requirements of Part 3.5D shall be measured for payment.

B. Each successful compression, tension and lateral pre-construction load test performed, including load frame and/or reaction piles, test pile, testing, and load test report, shall be considered a single unit price item.

C. Each successful compression, tension and lateral construction quick load test performed, including load frame and/or reaction piles, test pile, testing, and load test report, shall be considered a single unit price item.

4.2 PAYMENT

A. Each compression pile and each tension pile, approved and accepted by the Owner, shall be paid at the unit price indicated on the bid form.

B. Each successful pile load test, approved and accepted by the Owner, shall be paid at the unit prices indicated on the bid form.

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